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Effects of Curing Methods and Supplementary Cementitious Material Use on Freeze Thaw Durability of Concrete Containing D-Cracking Aggregates

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For concrete pavements in Kansas, the most effective method of increasing their sustainability is to increase the service life. One of the principle mechanisms of concrete pavement deterioration in Kansas is freezing and thawing damage. Some Kansas limestone aggregates are known to be very susceptible to D-cracking and have resulted in millions of dollars in maintenance costs. The Kansas Department of Transportation (KDOT) has developed rigorous testing procedures for limestone aggregate use in concrete. In this study, the role of concrete curing, mixture proportioning, and aggregate type on the freeze thaw durability of concrete beams tested using ASTM C 666 method B were investigated. This study found that long periods of soaking in lime water produced more damage during freezing and thawing than standard KDOT curing methods. Curing for shorter period of time in a lime water bath at 100°F however gave comparable freeze thaw results to that seen with the longer standard KDOT curing regime. Increasing the concretes resistance to water penetration can greatly increase the freeze thaw durability of concrete containing D-cracking susceptible aggregates. It was seen that the concrete volume of permeable voids and water absorption rate correlated well with the freeze thaw durability of concrete made with a poor quality aggregates. It is recommended that KDOT continue to enforce concrete permeability and volume of permeable voids specifications to improve the service life of Kansas concrete pavements.

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PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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Abstract

For concrete pavements in Kansas, the most effective method of increasing their sustainability is to increase the service life. One of the principle mechanism of concrete pavement deterioration in Kansas is freezing and thawing damage. Some Kansas limestone aggregates are known to be very susceptible to D-cracking and have resulted in millions of dollars in maintenance costs. The Kansas Department of Transportation (KDOT) has developed rigorous testing procedures for limestone aggregate use in concrete. In this study, the role of concrete curing, mixture proportioning, and aggregate type on the freeze thaw durability of concrete beams tested using ASTM C 666 method B were investigated. This study found that long periods of soaking in lime water produced more damage during freezing and thawing than standard KDOT curing methods. Curing for shorter period of time in a lime water bath at 100°F however gave comparable freeze thaw results to that seen with the longer standard KDOT curing regime. Increasing the concretes resistance to water penetration can greatly increase the freeze thaw durability of concrete containing D-cracking susceptible aggregates. It was seen that the concrete volume of permeable voids and water absorption rate correlated well with the freeze thaw durability of concrete made with a poor quality aggregates. It is recommended that KDOT continue to enforce concrete permeability and volume of permeable voids specifications to improve the service life of Kansas concrete pavements.

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Chapter 1: Introduction

1.1 Research Background

Sustainable roads provide safe, uninterrupted travel for the public on networks of roads and bridges that are economical and sensitive to the environment. For concrete pavements in Kansas, the most effective method of increasing their sustainability is to increase the service life. One of the principle mechanisms of concrete pavement deterioration in Kansas is freezing and thawing damage. Some Kansas limestone aggregates are known to be very susceptible to D-cracking and have resulted in millions of dollars in maintenance costs. The Kansas Department of Transportation (KDOT) has developed rigorous testing procedures for limestone aggregate use in concrete. For final qualification, the aggregate must pass KT-MR-22, Resistance of Concrete to Rapid Freezing and Thawing. The standard ASTM C 666 procedure B was modified by KDOT following a study in 1980 to require a 90-day cure period before the 300 freezing and thawing cycles, making the minimum time needed to perform the modified freezing and thawing test 5 months (Clowers 1999). Recently, KDOT has increased the number of cycles required to 660 cycles.

Besides a more rapid detection of deleterious aggregates, concrete pavement sustainability can be improved by changing mixture proportions. A recent study performed at Iowa State University showed that the concrete resistance to freeze thaw damage could be increased by reducing the concrete permeability (Wang, Lomboy, and Steffes 2009). Another study found that a small reduction in the water-cementitious material ratio (w/cm) from 0.44 to 0.4 increased the durability factor of four different Minnesota aggregates by an average of 14 (Snyder and Janssen 1999). It is unknown however how much of an increase in the freeze-thaw resistance of Kansas pavements can be gained by decreasing the porosity and permeability.

1.2 Problem Statement

KDOT has developed rigorous standards for limestone aggregates for use in pavements, including that they pass the KT-MR-22 concrete prism freezing and thawing test, which can take up to 6 months to complete. This project aimed to 1) shorten significantly the total time needed to perform the KT-MR-22 test method by changing the curing methods used and 2) determine any change in concrete freeze-thaw resistance as a result of a change in the concrete permeability from the use of supplementary cementitious materials (SCMs).

1.3 Research Objectives

The research objectives of this study were as follows:

- Freeze thaw testing was conducted on concrete made from the same aggregate source but with different amounts of wet curing at elevated temperatures to determine if accelerated curing with no drying period would give comparable durability to beams tested using the standard KDOT curing method of 67 days in the 100% moisture room, 21 days in a 50% relative humidity room, 2 days in a tempering tank, and finally 24 hours at 40°F.
- To determine any increase in freeze-thaw durability of concrete containing poor quality aggregates through a decrease in concrete water absorption and permeability by the use of supplementary cementitious materials (SCMs).

1.4 Scope of Research

This research project is composed of three tasks. Task1 focused on comparing the freeze thaw testing procedures and laboratory equipment between KDOT and KSU's laboratories. Three sets of beams were fabricated during Task 1. Of these three sets, concrete beams were cast in the field, cast at KSU, and cast at KDOT. Each casting included six beams, three for KDOT and three for KSU. The beams cast by KSU used the same aggregate as the beams cast in the field. Beams cast by KDOT contained aggregates with a history of poor performance in freezing and thawing conditions. Both the beams tested by KSU and KDOT were tested using KT-MR-22 for 300 cycles of freezing and thawing.

In task 2 the effect of accelerating curing procedures in the KT-MR-22 test on freeze thaw durability was examined. Three different batches containing different aggregates were used to test whether or not the curing procedures could be reduced. Five different curing methods were used in this study using different amounts of time in lime water at 100°F, curing for 28 days in the 100% moisture room, and the standard KTMR-22 curing methods for comparison. After curing, each specimen was tested in rapid freezing and thawing.

The third task focused on testing whether SCMs could improve freeze-thaw resistance by lowering water absorption rates. Three different aggregates were tested using different combinations of supplementary cementitious materials. With 7 different mixture proportions and 3 different aggregate sources, a total of 21 different mixtures were tested in this task. The seven different mixture proportions used are as follows:

- Batch 1: 100% Portland Cement, 0.39 w/cm
- Batch 2: 100% Portland Cement, 0.45 w/cm
- Batch 3: 25% Oklahoma Class C Fly Ash, 0.39 w/cm
- Batch 4: 25% Slag, 0.39 w/cm
- Batch 5: Ternary Blend (12.5% Oklahoma Class C Fly Ash, 12.5% Slag), 0.39 w/cm
- Batch 6: 25% Class F Fly Ash, 0.39 w/cm
- Batch 7: 25% Jeffery Energy Class C Fly Ash, 0.39 w/cm

Each SCM and aggregate source was selected in consultation with KDOT. Three beams for each mixture were tested under 600 freezing and thawing cycles in this task. In addition to the beams, nine cylinders were made to measure the concrete's ability to resist water penetration.

Chapter 2: Literature Review

Concrete is the most widely used construction material around the world. It can be measured both by strength and durability. Present day concrete failures are not typically due to a lack of strength, but are caused by concrete lacking in durability. Deterioration that happens gradually over time and reduces the service life of concrete can become expensive if concrete is repaired or replaced prior to the end of the anticipated service life. Durability of concrete is a major concern in regions of the world that are exposed to harsh climates where freezing and thawing occurs. This is the case for much of the Midwest, including Kansas. The Kansas Department of Transportation (KDOT) has spent much of its resources working on durability issues with concrete due to freezing and thawing conditions.

2.1 D-Cracking

The durability of concrete refers to its ability to withstand deterioration due to harsh environmental conditions. These conditions can act alone or together and include heating and cooling, freezing and thawing, wetting and drying, chemical attacks, and abrasion. Of these conditions, freezing and thawing is a major concern for KDOT. Deterioration due to freezing and thawing causes D-cracking to occur in Kansas (Koubaa and Snyder 2001).

D-cracking is a type of freeze-thaw damage in concrete pavements that occurs because of poor quality coarse aggregates. After the aggregates become saturated, the coarse aggregate is vulnerable to deterioration during freezing and thawing cycles. When the water inside the course aggregates is frozen, pressure builds up inside of the coarse aggregate. If the internal strength of the coarse aggregate is lower than the pressure applied by the expansion of the water inside the coarse aggregate, the coarse aggregate will crack. After cracks form, the deterioration process accelerates because of wedging action in cracks and the increase in potential water availability (Snyder and Janssen 1999). Concrete exposed to freezing and thawing cycles also can have damage that starts at the interfacial transition zone (Koubaa and Snyder 2001). Poor air-entrainment can also contribute to freeze-thaw damage near the joints or increase the rate of D-cracking development.

D-cracking displays a particular cracking pattern in concrete. The cracks tend to be closely spaced and run parallel to the concrete joints and edges. Before the cracks are visible at the surface, they usually develop below the surface in a horizontal plane. The first visible cracks usually start at intersections of two joints or at the corners of the slab. Cracking will work its way from the joints to the interior of the slab as time passes (Snyder and Janssen 1999). Though D-cracking is typically associated with concrete pavements, it is also possible to have D-cracking in concrete structures (Koubaa and Snyder 2001). Typical structures affected by D-cracking include structures such as bridge piers and dams. These structures are vulnerable to cracking near the water line. The concrete a few feet above the water line can be saturated and experience freezing and thawing (Pigeon and Pleau 1995).

D-cracking typically occurs first at the concrete joint because water is likely to stay at joints longer, causing them to saturate first. Since the bottom layer of the pavement is typically the most saturated layer, D-cracking is likely to initially occur in this layer first. However, it is possible for deterioration to occur first in both the middle and top layers of the pavement depending on the local saturation conditions (Snyder and Janssen 1999). This can occur when the concrete does not crack at the joint location, allowing water to pond in the joint or if water cannot drain through the crack. Proper drainage is important to keep water away from the concrete and prevent saturation (Li, Pour-Ghaz, Castro, and Weiss 2011).

2.2 Freeze Thaw Durability

2.2.1 Concrete Properties

There are several variables within the concrete mixture design that affect concrete resistance to bulk deterioration in freezing and thawing conditions. These factors include: the water to cement ratio, aggregate characteristics, additives, the air void spacing factor, and the curing period (Pigeon, Pleau, and Aitcin 1986).

2.2.1.1 Air Entrainment

Air voids are known to help reduce the damage of concrete that occurs due to cycles of freezing and thawing. As the percentage of air voids is important to the durability of concrete, the size and distribution of these air voids are equally important. A well-structured air void system can help reduce the hydraulic pressures that are caused by freeze thaw cycles (Distlehorst and Kurgan 2007).

Air bubbles naturally occur due to the mixing process of concrete. There are two mechanisms that induce air void instability in fresh concrete. The first mechanism is explained by small air bubbles combining to form a larger air bubble due to diffusion of air. The second failure mechanism occurs when flow ruptures the air bubbles. This type of failure can be caused by actions such as vibrating the concrete (Du and Folliard 2005). Air entrainment admixtures are used to stabilize air voids when they naturally form. The air entraining admixture molecules have both a hydrophilic and hydrophobic end. The hydrophilic ends of the molecules are towards the outside of the air bubble, normally with a negative charge. This negative charge on the outside of the air bubble is attracted to the positive charge of the cement grains. By forming a shell around the air bubble, the void is stabilized (Pigeon and Pleau 1995).

Damage due to freezing and thawing will be reduced if the entrained air voids are small and closely spaced. The spacing of the air voids needed to avoid damage is identified as the critical spacing factor (Pigeon and Pleau 1995). The spacing factor signifies average maximum distance in the cement paste from an air void (ASTM C 457 2012). The spacing factor increases as the voids are farther apart, and it decreases as the voids are spaced more closely together (Distlehorst and Kurgan 2007). A spacing factor of 200 μm is a common guideline for properly spaced air voids (Pigeon and Pleau 1995). To test for the percentage of air voids in concrete the pressure (ASTM C 231 2012), volumetric (ASTM C 173 2012), or gravimetric method (ASTM C 138 2012) may be used. These test methods fail to test the important spacing of the air voids. The size and spacing of the air void structure can be measured by microscopic examination of the hardened concrete or by the air void analyzer (AVA) on fresh concrete. The AVA measures the air void size distribution of mortar by mixing the mortar with a viscous liquid at the bottom of a column of water. The smaller bubbles take longer to float to the top of the column where the bubbles are collected and measured. The AVA, which was developed in the early 1990s, makes it possible to measure the air content and the spacing factor while the concrete is still fresh. (Distlehorst and Kurgan 2007). Microscopic examination of hardened concrete is expensive and is typically only done if the concrete begins to fail.

2.2.1.2 Water to Cement Ratio

Lower water-to-cement (w/cm) ratios have been shown to produce higher strength and freeze-thaw durability in laboratory tests. An increase in the concrete tensile strength will increase the stresses required to induce cracking during freezing. Lower w/cm also reduces the amount of water remaining after hydration which reduces the degree of saturation. Concrete with a lower w/cm will have a smaller pore size distribution and lower water permeability. This decrease in permeability slows down water absorption which will increase the time it takes to saturate the concrete and increase the durability of the concrete (Snyder and Janssen 1999). This benefit is limited to concrete that is intermittently exposed to moisture since concrete that is sufficiently saturated will show damage even with low w/cm.

2.2.1.3 Aggregate Quality

Coarse aggregates differ in physical and chemical properties based on the geology of where they are quarried. Argillaceous carbonate aggregates are limestone and dolomites that are made of at least 10 percent silt and clay particles. Since there is a larger volume of small pores in clayey argillaceous material than silty argillaceous material, clayey argillaceous material is generally thought to be less durable than silty argillaceous material. The clayey material in limestone aggregates absorbs extra water causing the aggregate to become saturated more rapidly and expand more. Argillaceous aggregates in Indiana were found to be nondurable if they contain more than 20 percent clayey or silty material (Shakoor, West, and Scholer 1982).

Argillaceous aggregates differ from non-argillaceous aggregates in a few important ways. Argillaceous material can be distributed throughout aggregates three different ways. It can be present via uniformly distributed fine particles. Argillaceous material can also take the form of thin irregular streaks or small elongated flakes. It is believed that argillaceous limestone aggregates have a higher tendency for durability issues when they are subjected to cycles of freezing and thawing. A previous study showed that poor quality argillaceous aggregates in Indiana tend to have a bulk specific gravity under 2.5 and an absorption capacity of more than 4 percent (Shakoor, West, and Scholer 1982). These durability trends are not universal and have not found to necessarily be predictive of performance in Kansas, making it difficult to develop aggregate qualification tests across geologic regions.

2.2.1.4 Size of Aggregates

Reducing the size of marginal quality coarse aggregates has shown to increase the life of concrete on-grade subject to freezing and thawing. The smaller aggregates allow water to escape faster which reduces the dilation of the aggregate during freezing from water freezing in the pores and the resultant stresses on the paste near the aggregate. Reducing the size of the aggregates will not make the concrete last forever, but the longer concrete remains durable the more money will be saved by state agencies (Chapin and Dryden 2001).

2.2.1.5 Curing

Typically concrete will become stronger as the hydration process is allowed to progress. The durability of the concrete due to freezing and thawing will likewise increase with increased levels of hydration. Concrete that is frozen before reaching 500 psi or experiences multiple freeze-thaw cycles before reaching 3500 psi will experience significant strength loss which is irrecoverable. This is because in low strength concrete large ice crystals will form causing large amounts of porosity and damage in the concrete microstructure (ACI 306 2010).

2.2.2 Saturation

For freeze thaw damage to occur the saturation of the concrete must reach a certain level, termed the critical degree of saturation. If the degree of saturation is below this critical degree of saturation, freeze-thaw damage may not occur. Conversely, even with air entrainment damage can occur after just one freeze-thaw cycle if the concrete is fully saturated (Li, Pour-Ghaz, Castro, and Weiss 2011). The critical degree of saturation is not a precise number and depends on many factors such as porosity, pore size distribution, and permeability. For three different mortars prepared with between 6 and 14% air by volume, the critical degree of saturation was found to be 86 to 88% (Li, Pour-Ghaz, Castro, and Weiss 2011).

Concrete that has been allowed to dry will usually have higher durability under freeze-thaw conditions because of a lower amount of freezable water in the pores. If the concrete is allowed access to moisture again however the concrete can re-saturate and experience freeze-thaw damage. Drying causes the pores in the concrete to become enlarged and to become more interconnected which increases permeability. This increased water permeability will allow water to reenter the concrete faster than before the drying period. This means that if the concrete is exposed to water for a long period of time after the drying period the benefits of the drying will disappear (Pigeon and Pleau 1995).

2.3 Scaling

Cycles of freezing and thawing can cause damage to concrete. This damage comes in both the form of scaling and internal cracking. Scaling and internal cracking do not always occur simultaneously. One form of deterioration can occur without the other (Pigeon, Pleau, and Aitcin 1986).

Unlike internal cracking, which is clearly defined by its' name, scaling only appears on the surface of the concrete. Scaling can occur when concrete freezes in water. Scaling is made worse by the application of deicer salts (Pigeon, Pleau, and Aitcin 1986). Other factors can cause scaling to take place:

> "Excessive bleeding, bad finishing procedures, plastic shrinkage cracking, overworking of the surface during the finishing operations, lack of curing, and early exposure to relatively high temperatures can all weaken the concrete surface and be an indirect cause of rapid scaling when concrete is exposed to freezing in water with or without de-icer salts being present (Pigeon and Pleau 1995)."

Concrete with an effective air void system can reduce the occurrence of scaling (Pigeon and Pleau 1995). Proper air entrainment will improve the concrete performance in deicer salt scaling, but will not prevent scaling in concrete with a high w/cm. Scaling can be signified by a loss of mass to the concrete specimen during freeze-thaw cycles. (Pigeon, Pleau, and Aitcin 1986).

2.4 External Freeze Thaw Resistance

The concrete mixture proportions affect the freeze thaw durability of concrete. External factors, in addition to the concrete's properties, also play a role in the rate at which deterioration occurs. The rate of freezing and thawing, the duration of the freezing period, and the freezing temperature all have an external effect on the durability (Basheer and Cleland 2006).

2.4.1 Rate of Freezing

The rate of freezing affects the durability of the concrete. Typical freezing rates that occur naturally are much lower than the freezing rates that are allowed in ASTM C 666. As the freezing rate increases the concrete deteriorates more rapidly. This increase in the freezing rate causes the critical air void spacing to decrease, meaning the air voids must be closer together as the freezing rate increases to prevent damage from occurring. The higher freezing rate increases the hydraulic pressures within the pores. These increased hydraulic pressures cause more deterioration to occur with each cycle (Nokken, Hooton, and Rogers 2004). This has led to the criticism that the ASTM test is harsher than actual conditions. Even if ASTM C 666 is harsher than actual conditions, it is still useful for comparing concrete mixtures to one another (Nokken, Hooton, and Rogers 2004).

It is believed that ice formation causes deterioration due to cycles of freezing and thawing. The formed ice applies pressure which acts as the mechanism that cracks the concrete (Basheer and Cleland 2006). Ice will begin to form as temperatures drop below freezing and moisture is present in the concrete. If air voids are available, ice will form in the air voids. However, if there are not sufficient air voids available the ice formation will begin to cause damage to the concrete. Frost damage can occur in either the paste or the aggregates. Paste frost damage occurs when water in the saturated paste is not able to make its way to the air voids fast enough or with low enough hydraulic pressure to avoid damage. As the capillary pores are filled with water and the temperature drops below freezing, the water tries to travel through the porous body towards the air voids. If the hydraulic pressure exceeds the tensile strength of the paste cracking will occur. According to Darcy’s law of water flow through porous bodies, this hydraulic pressure is given as a required pressure for water to travel a certain distance in a certain amount of time. Thus, due to Darcy’s law of water through porous bodies, to prevent damage and reduce the pressure on the paste one of two things must happen. Either a decrease in the freezing rate or a decrease in the space between air voids must occur. As the freezing rate is reduced ice forms slower giving water more time to move towards air voids. Of the two solutions provided to reduce the possibility of damage occurring in the paste, only one solution can be controlled. The freezing rate is up to the weather; whereas, the air void spacing can be controlled using proper air entraining admixtures (Pigeon and Pleau 1995).

2.4.2 Duration of Freezing Period

Freezing periods generally last for longer time periods under actual freezing and thawing conditions than in the ASTM C 666 test. It would be hard to reproduce actual freezing and thawing cycles in a test, and would take too long to be practical. ASTM C 666 tests the concrete prisms at a shorter duration of freezing than actually occurs to speed up the test. The decrease in the freezing duration will decrease the amount of deterioration seen. These longer durations cause ice to continue to form gradually in layers during a freeze cycle (Nokken, Hooton, and Rogers 2004).

2.4.3 Freezing Temperature

The temperature that water freezes and the temperature that water freezes in the pores of concrete are different. The temperature that water freezes in concrete pores depends on the temperature, the pore size, and the pore distribution. As the size of the pores decrease, the temperature needs to be lower to freeze the water (Nokken, Hooton, and Rogers 2004). The Gibbs-Thomson equation used to predict this is shown as Equation 2.1:

$$\gamma_{CL} k_{CL} = \int_{T}^{T_{M(\infty)}} \frac{(S_L - S_C)}{V_L} d \qquad \textbf{Equation 2.1}$$

where γ_{CL} is the crystal/liquid interfacial energy, k_{CL} is the curvature of the crystal/liquid interface, S_L *and* S_C are the entropies of the liquid and crystal, respectively, $T_{M(\infty)}$ is the melting point of a macroscopic crystal, and V_L is the molar volume of the liquid (Sun and Scherer 2010). Freezing of all pores, including capillary pores, generally occurs by -10 degrees Celsius (Nokken, Hooton, and Rogers 2004). For all of the water to freeze during each cycle it is important that the temperature drops low enough so that ice forms in the small pores during testing. Small laboratory samples can behave differently from large pavements however. Having a greater probability for a nucleation to occur on a large scale field pavement, it should be expected to have some ice formation occurring at slightly higher temperatures than the lab data suggests (Sun and Scherer 2010).

2.4.4 Damage Measurement

2.4.4.1 Length Change

The length of the concrete specimen is optionally measured in the ASTM C 666 test (ASTM C 666 2008). This length change is a good measure for the internal damage that occurs in the concrete specimen and correlates well with dynamic modulus values (Pigeon, Pleau, and Aitcin 1986). Minor length changes in concrete specimens can be expected due to the difference in the coefficient of thermal expansion for the cement paste and the aggregates. These small length changes due to discrepancies in coefficients of thermal expansion will occur even in durable concretes. Neither the air void structure nor the quality of the course aggregate has bearing on this small expansion. Larger expansions indicate that cracking from freeze-thaw damage has occurred within the specimen. These cracks cause voids that expand the concrete (Pigeon, Pleau, and Aitcin 1986).

2.4.4.2 Dynamic Modulus

The measurement of dynamic modulus of elasticity is specified as part of ASTM C 666 to measure the concrete freeze-thaw deterioration. The dynamic modulus can be tested either by forced resonance or impact resonance. Both methods measure the concrete fundamental longitudinal resonant frequencies. The frequency will decrease as the concrete begins to crack internally because waves travel slower through a damaged medium. As the length of the concrete specimen increases, the measured frequency decreases. (Pigeon, Pleau, and Aitcin 1986).

The relative dynamic modulus sometimes has trouble indicating only internal cracking. As surface scaling occurs, the resonant frequency also decreases. The pulse velocity is affected differently by surface scaling and internal cracking. Surface scaling tends to deteriorate linearly; however, length change and internal cracking cause deterioration to proceed in an exponential manner (Pigeon, Pleau, and Aitcin 1986).

2.5 Preventative Measures

Several methods have been studied to lengthen the life of the concrete exposed to freezing and thawing conditions. Most of these studies have focused on reducing the saturation levels of the concrete and reducing the size of the aggregates.

2.5.1 Saturation Reduction Options

To prevent D-cracking from happening in regions that undergo freeze thaw cycles, one of the factors leading to D-cracking deterioration must be minimized or eliminated. Either good coarse aggregates should be used or if good coarse aggregates are not available, the concrete should not be allowed to become saturated with moisture. Good positive drainage of pavements will help reduce D-cracking in concrete by not allowing the concrete to become saturated with water. One study looked at the impact of the subbase layer beneath the concrete pavement in Ohio (Dryden and Chapin 2009). In this study subbase systems were prepared three different ways. One pavement was placed on granular material with longitudinal drains, another was placed on granular material without drains, and the final pavement was placed directly onto the clay subgrade. The pavement on the clay subgrade deteriorated quickly and the longitudinal pipe and the plain granular subbase showed similar results. In the past vapor barriers have been used with the hope of preventing moisture from penetrating up into the concrete slab. These vapor barriers were located between the subbase and the concrete. It was determined however that the vapor barrier does not increase the durability of the concrete pavement (Dryden and Chapin 2009). Just as the vapor barrier doesn’t allow water to enter the concrete, the barriers prevent water from leaving through the bottom of the pavement, keeping the concrete saturated.

2.5.2 Aggregate Blending

Coarse aggregates with good durability are not always readily available in regions susceptible to freeze thaw damage, as is the case in Kansas. There are a number of solutions that have been tried to increase the durability of specific aggregates. There are strong economic and political incentives to make coarse aggregates of poor durability suitable for use in concrete. By reducing the size of the coarse aggregate, freeze thaw damage is somewhat delayed. A study conducted by Snyder and Janssen (1999) showed that durability can be increased by blending good aggregates and poor aggregates. The blend of fifty percent poor and fifty percent good coarse aggregate is the highest percentage of poor coarse aggregate that has shown an improvement in the durability (Snyder and Janssen 1999). As this study was laboratory based, there are still questions about the economic viability and long term field durability of aggregate blending using different aggregate sources however.

2.5.3 Surface Treatments

Availability of good aggregates is sometimes scarce and costly. Many treatments have been tried in order to make poor aggregates perform better under freeze testing and allow their use in concrete. Surface treatments composed of silanes and siloxanes have been applied to highway structures to attempt to prevent the concrete from becoming saturated with water (Basheer and Cleland 2006). It is thought that the hydrophobic treatments line the concrete pores, repeling water away from the pores. Although this treatment does decrease the amount of water that saturates the concrete, it does not prevent water vapor from entering concrete. Since this treatment allows the passage of water vapor, the concrete is allowed to dry naturally. This treatment did show positive results. The surface treated concrete did not become critically saturated as soon as the concrete without the surface treatment (Basheer and Cleland 2006). Concrete pavements in the field however could still become saturated from the bottom up.

2.6 Supplementary Cementitious Materials

Supplementary cementitious materials (SCM) provide an environmentally friendly alternative to replace part of the cement used in concrete. Fly ash, slag, and silica fume are examples of commonly used SCMs that are industrial byproducts (Wang K. 2003).

There are some advantages to using SCM's other than their low cost and positive environmental impact. SCM's can improve the workability, durability, and long term strength of concrete. Heat of hydration is also typically lower for concretes using SCM's. This lower heat of hydration can be a benefit in reducing the risk for thermal cracking to occur in the concrete. To combat the low heat of hydration, accelerators or slab coverings may have to be used in cold weather situations (Wang K. 2003). Since SCM's have a slow heat of hydration, longer curing times may delay construction that is on a tight schedule needing concrete to meet early strength requirements (Ge and Wang 2005).

SCMs are thought to impact the freeze thaw durability of concrete by reducing the permeability to reduce water ingress and saturation and increase the long term concrete strength. Some SCMs such as fly ash can contain unburnt carbon. This unburnt carbon can absorb air entraining admixtures, increasing the dosage of air entraining admixture required to provide adequate protection against freezing and thawing conditions (Ley, Harris, Folliard, and Hover 2008). More research is needed to determine how SCM use can affect aggregate freeze thaw durability when used in concrete.

2.7 Summary

The durability of concrete is affected by many different factors. These factors include the concrete permeability, pavement subbase drainage, air void structure, aggregate quality, freezing rate and amount, and salt level. The extent to which the freeze-thaw durability of concrete containing D-cracking aggregates can be improved by reducing the concrete permeability is unknown and is the subject of this study.

Chapter 3: Concrete Batching, Curing, and Testing Procedures

3.1 Aggregate Characterization

The coarse aggregate specific gravity and absorption capacity was measured according to ASTM C 127 (ASTM C 127 2012). The coarse aggregate specific gravity and absorption was measured as follows:

1. The sample was soaked in water for 24 ± 4 hours.
2. The sample was surface dried by shaking the coarse aggregates in a large towel. Care was taken to avoid evaporation of water from within the pores.
3. This sample surface dry mass was recorded as B.
4. The sample was submerged in water at a temperature of 23 ± 2 ºC.
5. The submerged sample mass was recorded as C.
6. The sample was oven dried at a temperature of 110 ± 5 ºC.
7. The oven dried mass was recorded as A.

Equations 3.1 and 3.2 were used to calculate coarse aggregate specific gravity and the absorption capacity:

$$Specific\ Gravity\ (SSD) = \frac{B}{B - C}$$ **Equation 3.1**

$$Absorption\ (\%) = \frac{B - A}{A} \times 100$$ **Equation 3.2**

Where:

A = mass of oven dried sample (g)

B = mass of saturated- surface- dry sample in air (g)

C = mass of saturated sample submerged in water (g)

The fine aggregate specific gravity and absorption capacity were measured as follows (ASTM C 128 2012):

1. The fine aggregate was soaked in water for 24 ± 4 hours.

2. Excess water was drained and the fine aggregate was emptied on a nonabsorbent surface.
3. The fine aggregate was allowed to dry naturally and was continuously mixed to avoid unequal drying. Drying was complete when the fine aggregate reached saturated surface dry conditions. Saturated surface dry conditions were determined by the cone test:
    a. The cone was filled with fine aggregate.
    b. The fine aggregate in the cone was lightly tamped 25 times by dropping the tamper approximately 5 mm. This was done to consolidate the fine aggregate.
    c. The cone was removed.
    d. If the fine aggregate took the shape of the cone then drying was continued. The cone test was repeated until the fine aggregate started to slump as the cone was removed.
4. The gravimetric method was used to determine the specific gravity and the absorption capacity.
5. A 500 cm^3 pycnometer was partially filled by water followed by 500 ± 10 grams of saturated surface dry fine aggregate.
6. The exact mass of fine aggregate added to the pycnometer was recorded as S.
7. The pycnometer was agitated to remove the air bubbles.
8. Once there were no air bubbles the pycnometer was filled to capacity.
9. The weight of the pycnometer filled with water and fine aggregate was recorded as C.
10. The contents of the pycnometer were emptied to be oven dried.
11. The oven dry mass of the sample was recorded as A.
12. The pycnometer was filled with water and weighed; this mass was recorded as B.

Equations 3.3 and 3.4 were used to determine the specific gravity and absorption capacity of the fine aggregate:

$$Specific\ Gravity = \frac{S}{(B + S - C)}$$ **Equation 3.3**

$$Absorption\ Capacity\ (\%) = \frac{S - A}{A} \times 100$$ **Equation 3.4**

Where:

A = mass of the oven dried sample.

B = mass of the pycnometer filled with water.

C = mass of the pycnometer filled with water and the sample.

S = mass of the saturated surface dried sample added to the pycnometer.

3.2 Batching Procedures

Concrete batching was done in general accordance with ASTM C 192/ C 192M (ASTM C 192 2006). The following procedures were used:

1. The coarse and fine aggregate were stored at room temperature overnight to ensure consistent temperature.
2. The coarse aggregate moisture content was measured.
3. The required weight of the coarse aggregate and mix water were adjusted to account for the coarse aggregate moisture content.
4. The air entraining admixture was added to the mix water.
5. The base of the mixing pan was wiped with a damp towel.
6. The coarse aggregate was added to the mixer shown in Figure 3.1.
7. The mixer was started and the fine aggregate, cement/SCM, and water were added.
8. The mixer was run for three minutes after the concrete materials were added.
9. The mixer was stopped and allowed at rest for three minutes. The mixer was covered during the rest stage.
10. The mixer was finally uncovered, and the concrete was mixed for two additional minutes.

FIGURE 3.1
Concrete Mixer Used in This Study

3.3 Slump and Air Content Procedures

The concrete slump was measured according to ASTM C 143 (2010). Concrete used for the freeze-thaw test KTMR-22 is required to have a slump between 1.5 and 2.5 inches. An example of this is shown in Figure 3.2. The concrete air content was measured using the pressure method according to ASTM C 231 (2010). If the air content was not between 5 and 7 percent, the air entraining dosage was adjusted and the mixture remade. Figure 3.3 shows the air content being measured.

FIGURE 3.2
Concrete Slump Test

FIGURE 3.3
Concrete Air Pressure Meter
Used in Testing

3.4 Specimen Preparation

3.4.1 Specimen Fabrication

The research team visited the KDOT materials and research facility to observe concrete mixture proportioning, specimen preparation, and testing procedures to ensure that procedures used mirrored those use by KDOT.

4 x 8 inch concrete cylinders were made for testing the concrete's resistance to fluid movement through the concrete. The cylinders were filled using the rodding procedure described in ASTM C 192/ C 192M (ASTM C 192 2006). Concrete was placed in the cylinder in 2 lifts. Each lift was rodded 25 times followed by tapping 10–15 times with an open palm for each lift. 4 x 3 x 16 inch concrete prisms were made for freeze-thaw testing. Pins were embedded in the prism ends to facilitate prism length change measurements. The molds were completely filled with concrete and consolidated. The concrete prisms were finished with a wooden trowel.

3.4.2 Curing Process

For tasks I and III of this study, the concrete freeze-thaw specimens were cured using the standard KTMR-22 procedure. The standard KTMR-22 procedure calls for concrete to be placed in a moist room for 67 days followed by 21 days in a chamber at 50% relative humidity at 73°F. The beams were then placed in a water bath at approximately 70°F for 24 hours. The prisms were then cured in a water bath at 40°F. In this procedure, the concrete beams are cured for the extended period in the moist room to allow the paste to harden sufficiently. This is meant to ensure that the concrete did not incur damage during freezing and thawing cycles because of a weak paste system and only because of the aggregates. The drying period is intended to better simulate field concrete conditions where the concrete is typically allowed to dry naturally before water reabsorption during a precipitation event, although concrete pavement moisture level measurements in-situ are needed to verify if this intention matches pavement moisture conditions in Kansas. The concrete is finally soaked in water to resaturate the concrete and bring the concrete to the thaw temperature used to eliminate thermal effects on the length change measurements.

Different curing procedures were used in task II to test the effects of curing on the freeze-thaw resistance of concrete containing porous limestone aggregates. After batching the concrete all of the specimens were placed in the 100% moisture room. It was hypothesized that the paste hydration could be accelerated to have similar strength as to that for concrete cured using the standard KTMR-22 method. Curing procedures 1 to 3 used heat curing to accelerate the curing process. For these procedures, the concrete specimens remained in the moist room for 7 days. Concrete specimens for these three procedures were then transferred to a lime water bath at 100°F. Concrete cured using procedure 1 remained in the lime water bath for 7 days, using curing condition 2 for 14 days, and curing condition 3 for 21 days. Following the lime bath curing, these specimens were placed in a 40°F water bath for 24 hours prior to being placed in the freeze thaw machine for testing. Concrete specimens cured using method 4 remained in the moist room for 28 days prior to being placed in the 40°F water bath for 24 hours. This procedure without any drying period was performed and compared to the standard KTMR-22 curing method to determine if the extended moist curing and drying process affected the durability. Concrete cured according to method 5 was cured using the standard KTMR-22 curing process for comparison.

3.5 Laboratory Testing

Freezing and thawing cycles were performed according to ASTM C 666/C 666M procedure B (2008). Concrete specimens made as part of task three were tested using the water absorption test ASTM C 1585 (2013), rapid chloride permeability test ASTM C 1202 (2012), and the volume of permeable voids test ASTM C 642 (2013). These test methods are indicators of the concrete's ability to resist water ingress which is required to saturate the concrete and cause damage to the concrete during freezing and thawing cycles. All of these test methods use 2 inch thick concrete disks cut from 4 x 8 inch concrete cylinders.

3.5.1 Concrete Rapid Freezing and Thawing Testing

The 4 x 3 x 16 inch concrete prisms were subjected to cycles of freezing and thawing. Concrete prisms tested in task I and II were tested using 300 freeze-thaw cycles, while prisms tested in task III were tested for 600 cycles. Six hundred cycles were chosen for testing while it was still being considered to increase the number of cycles used in KTMR-22. Freeze-thaw testing was performed according to ASTM C 666 Procedure B using a computer controlled automatic temperature cycling chamber. Under this procedure, the concrete prisms were frozen while they are completely surrounded by air. After the specimens reach 0°F, water was introduced into the chamber to heat the specimens during the thawing cycle. One freeze thaw cycle consists of the chamber lowering the temperature from 40 to 0 ºF and then increasing the temperature from 0 to 40 ºF. At least 20% of the cycle time must be spent for thawing. At the end of the freeze-thaw segments the temperature at the center of the beams must be within 3 ºF of the 0 and 40 ºF respective freeze and thaw baselines (ASTM C 666 2008). The center temperature of the beams was monitored by the use of two dummy concrete beams with thermocouples embedded in the center of the beam. Specimen location in the freezer was rotated in order to prevent possible sample bias from location in the freezer.

The prisms were placed in the freeze thaw chamber following the curing period. They were measured for deterioration at least every 36 cycles until completion of the required 300 or 600 cycles. Figure 3.4 and 3.5 show the freeze thaw machine and inside its chamber. To quantify the concrete degradation, the concrete mass, length change, and fundamental transverse frequency were measured. The fundamental transverse frequency reading was measured using the impact resonance method as shown in Figure 3.6. The prisms absorb water and absorbed water will freeze during the freezing segment of cycles. Freezing of water causes significant expansion in concrete prisms which leads to cracking. Water absorption and expansion are the reason for mass change and length change in concrete prism. As micro-cracks grow in the concrete prisms during expansion the transverse frequency will also decrease. The foam pad used in the study to place the concrete prisms on during testing was chosen to match that used by KDOT. The foam was approximately 50 mm thick as called out by KT-MR-22. Prisms were marked 25 mm from each end as shown in Figure 3.6. A circle was marked on the end that the accelerometer was placed and the x on the other side struck by the impactor. When the impactor struck the concrete a wave was sent through the concrete prism that was read by the accelerometer. The durability factor was calculated using Equation 3.5 and 3.6:

$$P_c = (\frac{n_1^2}{n^2}) \times 100$$ **Equation 3.5**

Where:

P_c = relative dynamic modulus of elasticity, after c cycles of freezing and thawing, (%)

n = fundamental transverse frequency at 0 cycles

n_1 = fundamental transverse frequency after c cycles

$$DF = \frac{PN}{M}$$ **Equation 3.6**

Where:

DF = durability factor

P = relative dynamic modulus of elasticity, at N cycles, (%)

N = number of cycles for which the test is discontinued or number of cycles that the test is to be terminated, whichever is less

M = number of cycles that the test is to be terminated

The concrete prism length change was calculated using Equation 3.7:

$$L_c = \frac{(l_2 - l_1)}{L_g} \times 100$$ **Equation 3.7**

Where:

L_c = length change after c cycles of freezing and thawing, (%)

l_1 = length reading at 0 cycles

l_2 = length reading after c cycles

L_g = effective gage length

FIGURE 3.4
20 Block Concrete Freeze Thaw Machine Used

FIGURE 3.5
Interior of Concrete Freeze Thaw Chamber

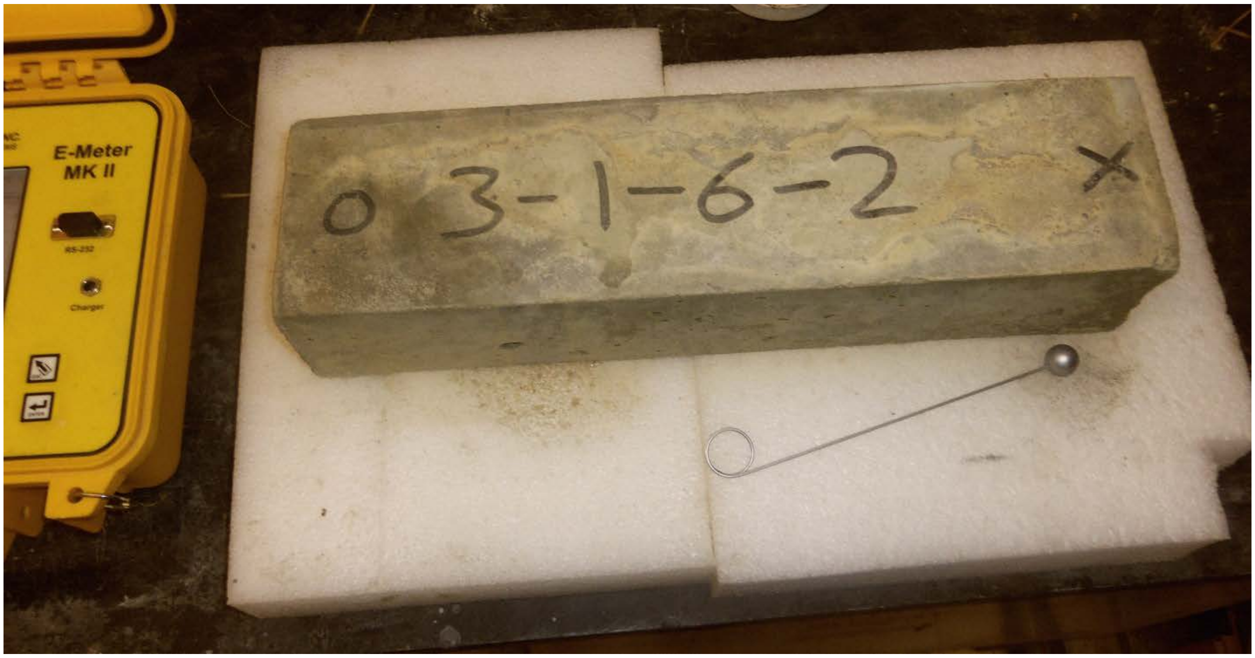

FIGURE 3.6
Transverse Frequency Setup Used in the Study

3.5.2 ASTM C 1585: Standard Test Method for Measurement of Rate of Absorption of Water by Hydraulic-Cement Concretes

The following steps were followed as prescribed by ASTM C 1585 (2013):

1. The 2 inch thick, 4 inch diameter specimens were placed above a saturated solution of potassium bromide in a desiccator. The desiccator was maintained at a temperature of approximately 122°F. The samples were maintained above the potassium bromide solution for three days. The specimens were prevented from contacting the potassium bromide solution.
2. Using a separate sealed container for each specimen, the specimens were stored at room temperature for 15 days to allow the moisture levels in the specimen to equilibrate.
3. Each specimen was weighed after the 15 day storage in the sealed container.
4. The diameter of the specimen was measured.
5. The sides and top of the specimens were sealed with plastic and waterproof tape.
6. The container was filled with room temperature water to a level that was 1–3 mm above the bottom of the specimens.

7. The mass of the sealed specimens was recorded at the following times:
    a. Initial weight
    b. 60 ± 2 seconds
    c. 5 minutes ± 10 seconds
    d. 10 ± 2 minutes
    e. 20 ± 2 minutes
    f. 30 ± 2 minutes
    g. 60 ± 2 minutes
    h. Every hour up to 6 hours ± 5 minutes
    i. Once a day for 8 days ± 2 hours
8. The initial absorption was calculated as the slope of a best fit line to the absorption data between 1 min and 6 hours. The secondary rate of absorption was calculated as the slope of a best fit line to the absorption data between 1 and 7 days.

3.5.3 ASTM C 1202: Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration

The concrete rapid chloride permeability was tested according to ASTM C 1202 (2012). The concrete electrical conductivity is dependent on the pore structure of the concrete. Electricity is conducted in concrete primarily through the pore solution. Larger, more connected and more numerous pores means more interconnected pore solution and a higher electrical charge that will be conducted in the test. Concrete pore structure can be affected by multiple factors including: the mix design, the degree of hydration, curing conditions, the use of SCMs, and construction practices (Joshi and Chan 2002). The following procedure was used to measure the charge passed through concrete according to ASTM C 1202 (2012):

1. Specimens were placed in a vacuum below 50 mm Hg. The vacuum was maintained for 3 hours.
2. The vacuum dessicator was filled with water that was boiled and allowed to cool to room temperature. The vacuum of the water/specimen filled container was maintained at a pressure below 50 mm Hg for 1 hour.

3. The vacuum was released, after which the specimens were left in the water-filled container for 18 ± 2 additional hours.
4. The specimens were sealed in the voltage cell using rubber gaskets and pvc pipe as shown in Figure 3.7.
5. 3.0 % NaCl solution was added to one cell while a 0.3 N NaOH solution was added to the other cell.
6. The wires were attached to the terminals. The NaOH was connected to the positive power supply and the NaCl was connected to the negative power supply.
7. The current was read initially when the power supply was turned on, and reading continued every 30 minutes for the duration of 6 hours.

   Equation 3.8 was used to determine the charge passed through the concrete:

$$Q = 900(I_0 + 2I_{30} + 2I_{60} + \ldots 2I_{300} + 2I_{330} + I_{360}) \qquad \textbf{Equation 3.8}$$

Where:

Q = charge passed (coulombs)

I_o = current immediately after voltage is applied (amperes)

I_t = current at t min after voltage is applied (amperes)

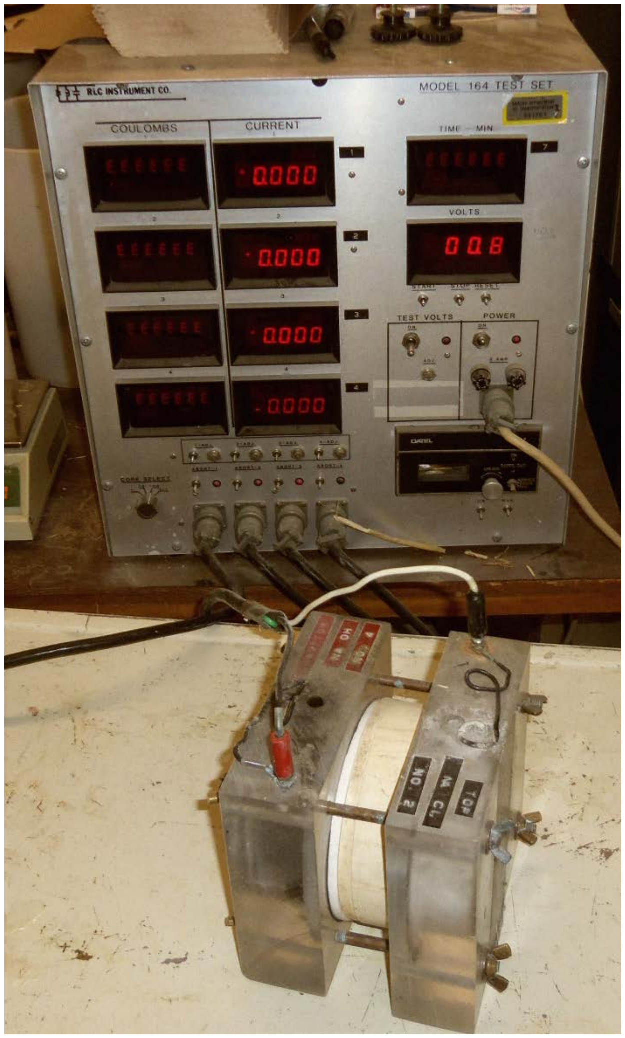

FIGURE 3.7
Rapid Chloride Permeability Test Setup

The chloride ion penetrability of the concrete can be classified based on the charge passed according to Table 3.1.

TABLE 3.1
Charge Classification Table (ASTM C 1202 2012)

Charge Passed (Coulombs)	Chloride Ion Penetrability
>4,000	High
2,000-4,000	Moderate
1,000-2,000	Low
100-1,000	Very Low
<100	Negligible

3.5.4 ASTM C 642: Standard Test Method for Density, Absorption, and Voids in Hardened Concrete

ASTM C 642, or the boil test, measures the density, absorption, and volume of permeable voids in concrete (ASTM C 642 2013). The following procedures were used to test the concrete volume of permeable voids:

1. The 2 inch specimens were oven dried for at least 24 hours.
2. The specimens were allowed to cool to room temperature before measuring mass.
3. Steps 1 and 2 were repeated until successive mass values were within 0.5 %.
4. The mass was recorded as A.
5. The specimens were submerged in room temperature water for at least 48 hours.
6. The specimens were then surface dried prior to determining the mass.
7. Steps 5 and 6 were repeated until successive mass readings within 24 hours of one another were within 0.5%.
8. The specimens were then boiled for 5 hours in such a way that the specimens were not resting on the bottom of the container.
9. The mass was recorded as C.
10. The water and specimens were allowed at least 14 hours to return to room temperature.
11. The mass was recorded as D.

Equation 3.9 was used to determine the volume of permeable pore space:

$$Volume\ of\ permeable\ pore\ space\ (\%) = \frac{(C-A)}{(C-D)} \times 100$$ **Equation 3.9**

Where,

A = mass of oven-dried sample in air (g)

C = mass of surface-dry sample in air after immersion and boiling (g)

D = mass of sample in water after immersion and boiling (g)

Chapter 4: Materials

4.1 Coarse Aggregate

Concrete used in task 1 contained aggregate used in concrete for construction of the K-18 Ogden to Manhattan Improvement Project. Concrete samples were made on-site with fresh concrete during construction.

Four different limestone coarse aggregates were used for task II and III of this project. The aggregates are labeled for organizational purposes as A, B, C and D. Aggregates A, B, and C were used in task II, whereas aggregates A, C, and D were used in task III. Aggregate A was collected from the Moline Quarry, aggregates B and C were collected from different beds from the Ottawa Quarry, and aggregate D was collected from a concrete batch plant in Topeka. These aggregates were selected by KDOT using past test results and field experience as a guideline. Aggregate A passed both KDOT's laboratory testing. Additionally, aggregates from that quarry had a good field history. Both aggregate C and D have shown failures in previous lab testing, whereas aggregate B was considered to be a marginal aggregate.

The gradation of the coarse aggregate used was specified by the KT-MR-22 test. Fifty percent of the coarse aggregates used were retained on the 3/8 inch sieve and passing the ½ inch sieve. The remaining coarse aggregates used were retained on the ½ inch sieve and passing the ¾ inch sieve. The coarse aggregates were sieved to achieve proper gradation prior to mixing the concrete. The coarse aggregate specific gravity and absorption capacity testing are shown in Table 4.1.

TABLE 4.1
Specific Gravity and Absorption Capacities of the Coarse Aggregates

	A	B	C	D
Specific Gravity	2.76	2.60	2.61	2.61
Absorption Capacity %	2.58	2.72	3.00	3.32

4.2 Fine Aggregate

The fine aggregate used was Kaw River sand as specified by the KT-MR-22 test. The specific gravity and absorption capacity of the fine aggregate were determined following ASTM C 128 (2012). Results from this test are shown in Table 4.2.

TABLE 4.2
Specific Gravity and Absorption Capacity of the Fine Aggregate

	Fine Aggregate
Specific Gravity	2.5
Absorption Capacity	0.65

4.3 Cement

The task I beams were cast with concrete sampled from the K-18 Ogden to Manhattan Improvement Project. All of the remaining concrete used in this project was made with a Type I/II cement produced by Monarch Cement. The cement was chosen as specified by the KT-MR-22 test which requires the cement to meet Type II classification and be produced by Monarch Cement. The composition of the cement used is given in Table 4.3.

TABLE 4.3
Cement Composition

	Content
SiO_2 (%)	21.9
Al_2O_3 (%)	4.3
Fe_2O_3 (%)	3.4
CaO (%)	63.7
MgO (%)	2.0
Na_2O (%)	0.2
K_2O (%)	0.5
Na_2O_{eq} (%)	0.5
SO_3 (%)	2.6
LOI (%)	0.5
Free CaO (%)	0.9
C_3S (%)	51.7
C_2S (%)	23.8
C_3A (%)	5.5
C_4AF (%)	10.4

4.4 Supplementary Cementitious Materials

These materials were selected by KDOT based on availability for possible future usage. Amongst the group of SCMs selected are two Class C fly ashes, one Class F fly ash, and slag cement. Table 4.4 shows the SCM chemical and physical properties.

TABLE 4.4
Supplementary Cementitious Material Composition and Properties

	Fly Ash C1	Fly Ash C2	Fly Ash F1	Slag Cement
SiO_2 (%)	36.37	28.18	52.1	-
Al_2O_3 (%)	20.13	21.02	18.28	-
Fe_2O_3 (%)	7.03	5.58	6.41	-
CaO (%)	24.13	29.89	12.93	-
MgO (%)	5.15	7.79	2.68	-
Na_2O (%)	1.71	-	0.58	0.83
K_2O (%)	0.51	-	0.84	-
SO_3 (%)	1.25	2.72	1.99	
LOI (%)	0.22	0.17	0.54	-
Moisture Content (%)	0.06	0.07	0.11	-
LOI (%)	0.22	0.17	0.54	-
Specific Gravity	2.61	2.78	2.49	2.98

4.5 Admixture

The only air entraining admixture used in this project was Daravair 1000, an air entraining admixture. Daravair is made principally of a high-grade saponified rosin (W.R. Grace and Co. 2009).

4.6 Mixture Proportions

The mixture proportions used is shown in Table 4.5. The following procedures were used to batch the concrete:

TABLE 4.5
Theoretical Mixture Proportions

Concrete Mixture	Cement (lb/yd^3)	SCM 1 (lb/yd^3)	SCM 2 (lb/yd^3)	Water (lb/yd^3)	CA (lb/yd^3)	FA (lb/yd^3)	Admixture (oz/yd^3)
Task 2							
Aggregate A	602	-	-	235	1547	1547	5.54
Aggregate B	602	-	-	235	1503	1503	5.54
Aggregate C	602	-	-	235	1506	1506	5.54
Task 3							
Aggregate A							
0.39 w/cm	602	-	-	235	1547	1547	5.54
0.44 w/cm	602	-	-	265	1507	1507	5.54
C1	451	150	-	235	1534	1534	6.92
Slag	451	150	-	235	1544	1544	5.54
Ternary Blend (C1 & Slag Cement)	451	75	75	235	1539	1539	6.57
F1	451	150	-	235	1530	1530	7.27
C2	451	150	-	235	1538	1538	6.92
Aggregate C							
0.39 w/cm	602	-	-	235	1506	1506	5.54
0.44 w/cm	602	-	-	265	1466	1466	5.54
C1	451	150	-	235	1493	1493	6.92
Slag	451	150	-	235	1502	1502	5.54
Ternary Blend (C1 & Slag Cement)	451	75	75	235	1497	1497	6.57
F1	451	150	-	235	1489	1489	7.27
C2	451	150	-	235	1497	1497	6.92
Aggregate D							
0.39 w/cm	602	-	-	235	1506	1506	5.54
0.44 w/cm	602	-	-	265	1466	1466	5.54
C1	451	150	-	235	1493	1493	6.92
Slag	451	150	-	235	1502	1502	5.54
Ternary Blend (C1 & Slag Cement)	451	75	75	235	1497	1497	6.97
F1	451	150	-	235	1489	1489	7.27
C2	451	150	-	235	1497	1497	6.92

Chapter 5: Results

5.1 Task 1

The aggregates tested from concrete specimens made from concrete containing the aggregate used on the K-18 Ogden to Manhattan Improvement Project showed no damage during the freeze thaw testing. The concrete tested by KSU and KDOT showed very similar relative modulus during testing for these beams as shown in Figure 5.1. Figure 5.2 shows the relative modulus of the beams that were cast by KDOT and sent to KSU using an aggregate of poor quality. There was a difference in the rate of deterioration of concrete beams tested between KDOT and KSU tested beams. ASTM C 666 (2008) contains the following statement about comparing freeze-thaw results from different laboratories: “No data are available for mulitlaboratory precision. It is believed that a multilaboratory statement of precision is not appropriate because of the limited possibility that two or more laboratories will be performing freezing-and thawing tests on the same concretes.” ASTM C 666 states that the acceptable range of concrete durability factors from tests conducted at the same laboratory with average values between 50 and 70 is 32.9. At 312 cycles, the difference in average durability factors for tests conducted at KSU and KDOT was 34.4, only slightly higher than that expected for tests conducted at the same laboratory. Since multi-laboratory precision values are almost always larger than single-labortory precision statements, the results found between the two laboratories should not be considered unacceptable. To help confirm that the equipment being used was functioning properly, KSU researchers traveled to KDOT to verify that the impact resonance equipment being used by KSU gave similar results to that being used by KDOT. It was found that the equipment being used by KSU gave nearly identical resonant frequencies as measured by KDOT.

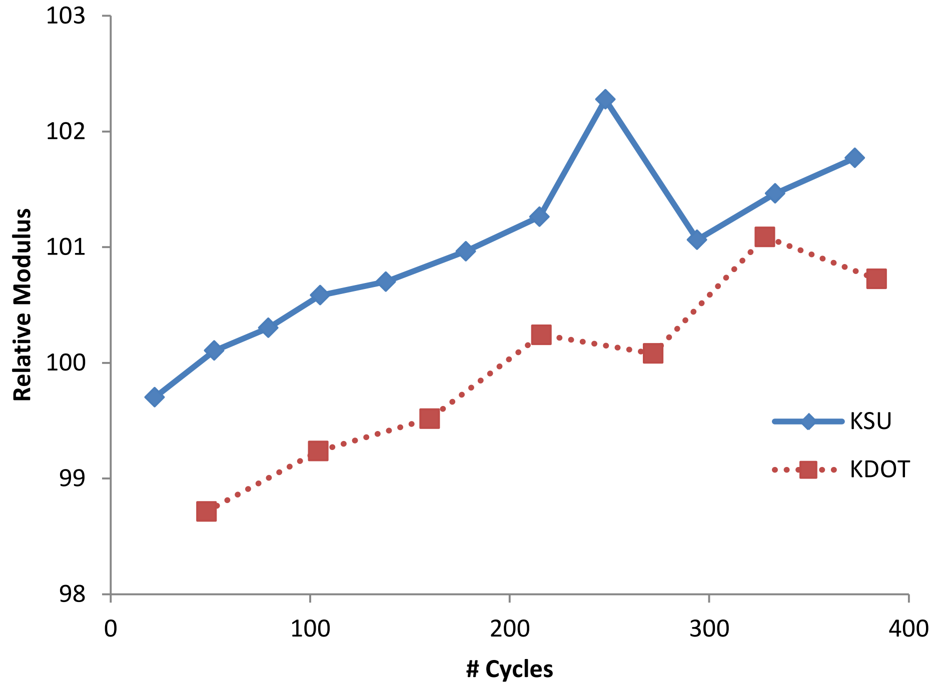

FIGURE 5.1
Relative Modulus During Freeze Thaw Testing for Concrete Specimens Made from Concrete Sampled from the K-18 Ogden to Manhattan Improvement Project

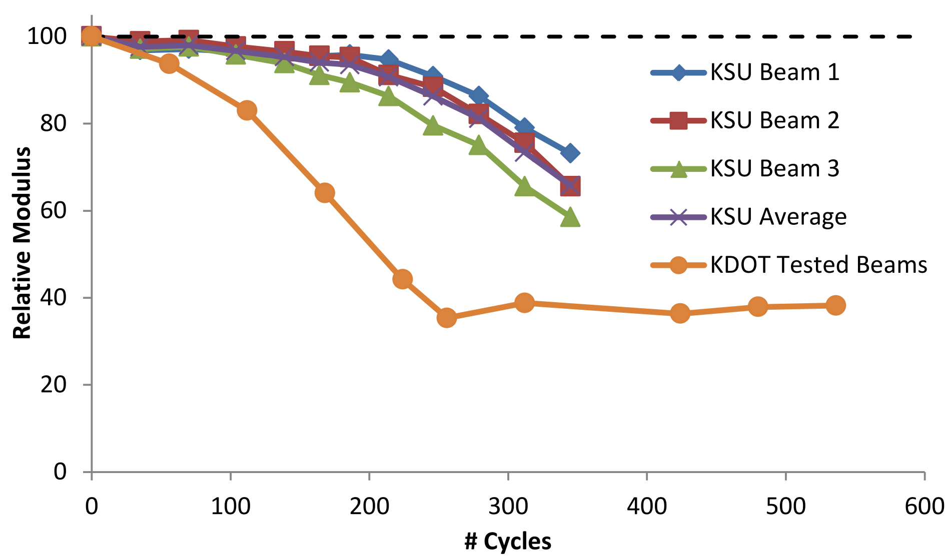

FIGURE 5.2
Task 1 Freeze Thaw Results for Beams Made by KDOT

5.2 Task 2

Task 2 of the project was performed with mixtures made with aggregates A, B, and C. The five different procedures included three that was cured for part of the time in a lime water bath at 100°F, one that remained in the moisture room, and one that underwent the standard KT-MR-22 curing process. Figures 5.3 through 5.5 show the relative modulus measurements for aggregates A, B, and C, respectively. Figure 5.6 through Figure 5.8 shows the length change data for aggregates A, B, and C, respectively during the freeze thaw cycles. Figures 5.9 through 5.13 show the concrete freeze thaw results grouped by curing method used. The results when grouped by curing method showed that aggregate A performed well in freeze-thaw testing, with aggregate C showing poor performance, as expected from previous experience with these quarries by KDOT. This also shows that removing the drying period from the curing period could help to better differentiate poor from well performing aggregates in fewer freeze-thaw cycles.

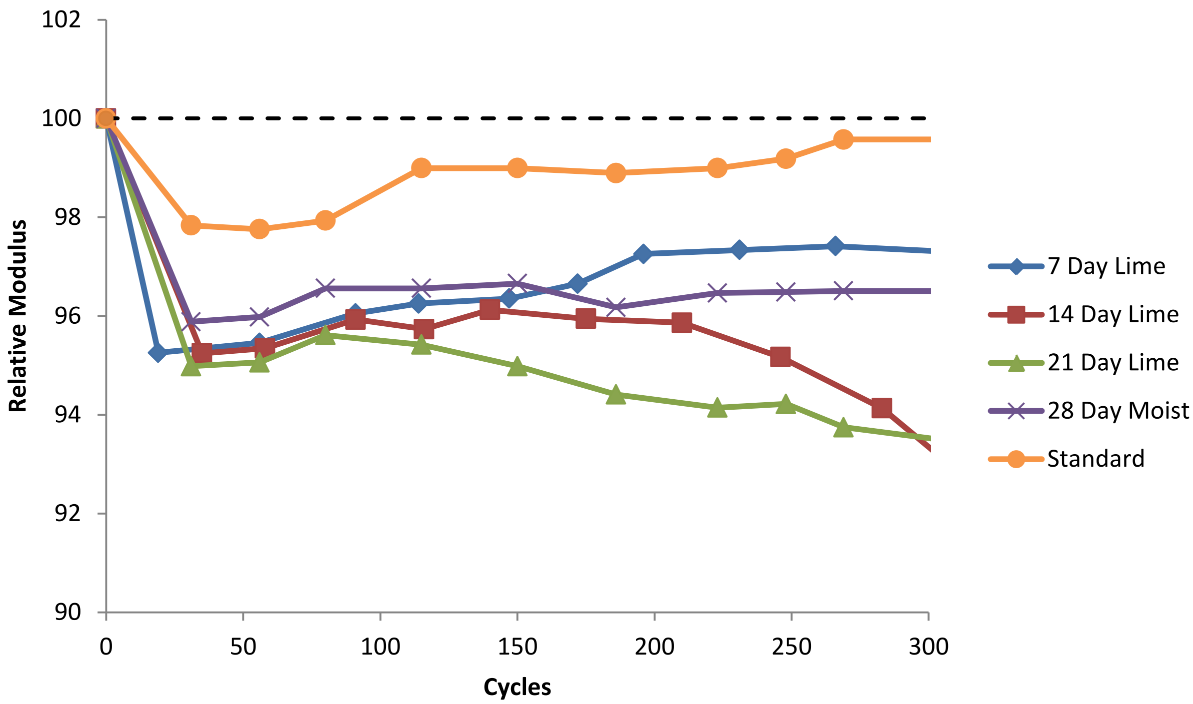

FIGURE 5.3
Aggregate a Relative Modulus with Different Curing Methods

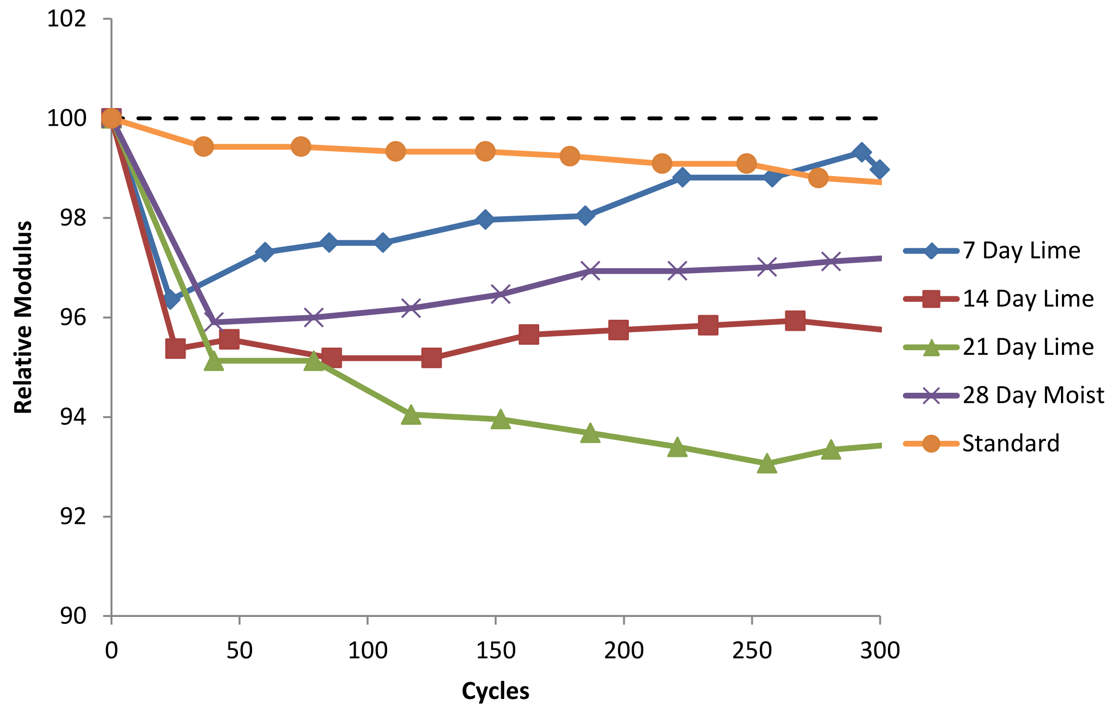

FIGURE 5.4
Aggregate B Relative Modulus with Different Curing Methods

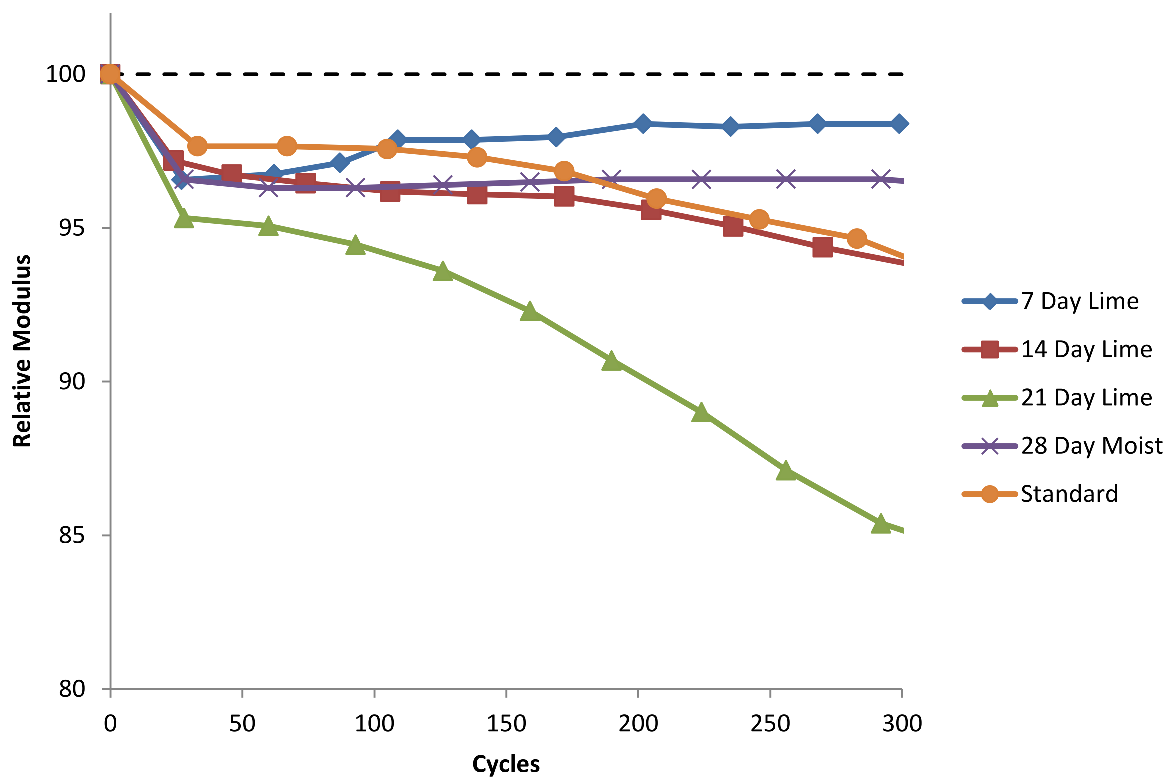

FIGURE 5.5
Aggregate C Relative Modulus with Different Curing Methods

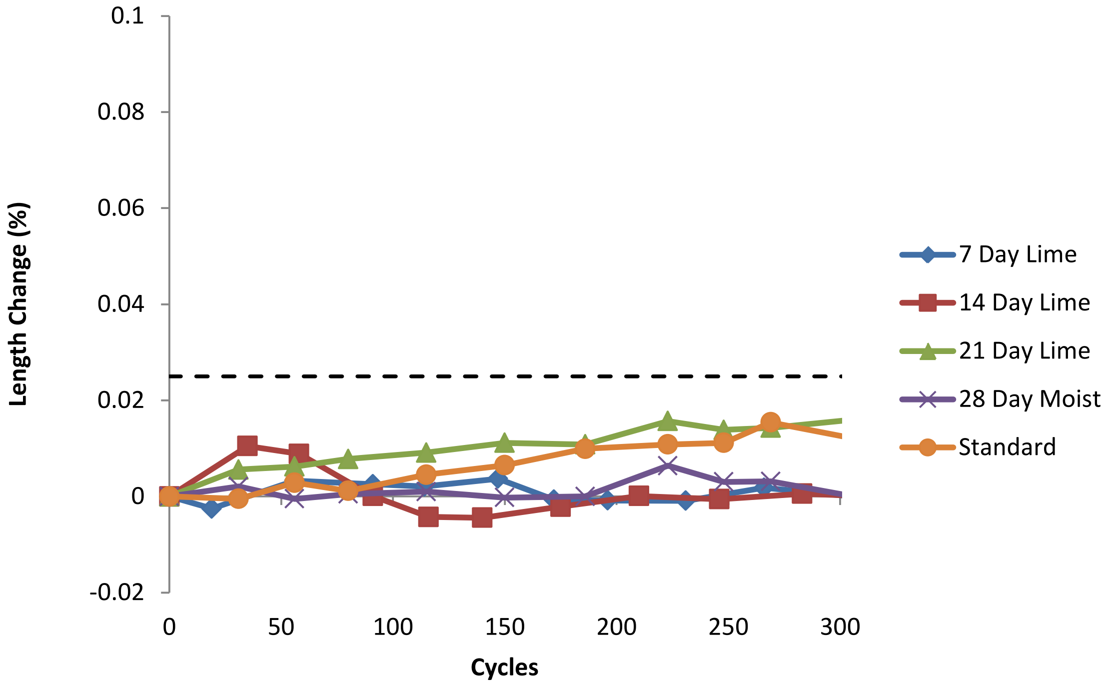

FIGURE 5.6
Aggregate A Length Change with Different Curing Methods

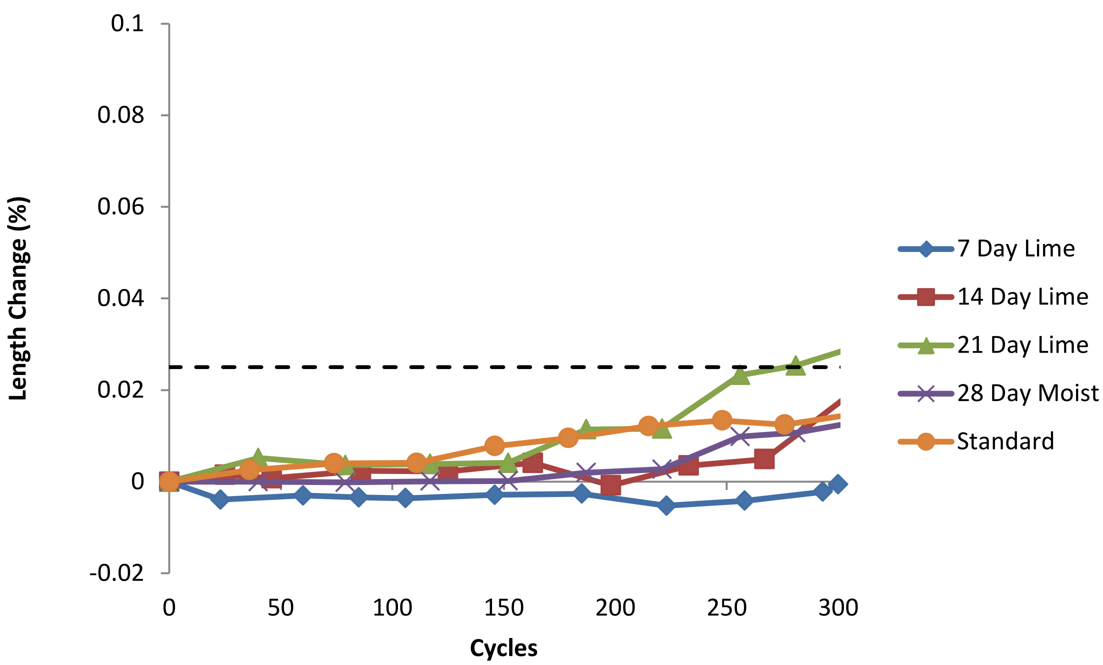

FIGURE 5.7
Aggregate B Length Change with Different Curing Methods

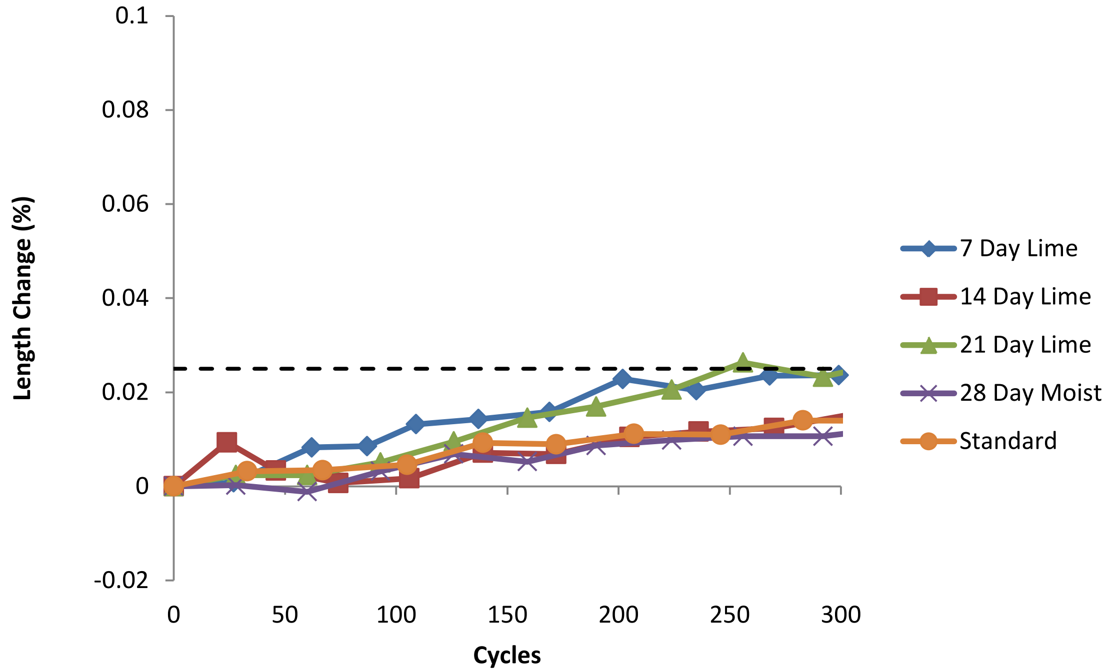

FIGURE 5.8
Aggregate C Length Change with Different Curing Methods

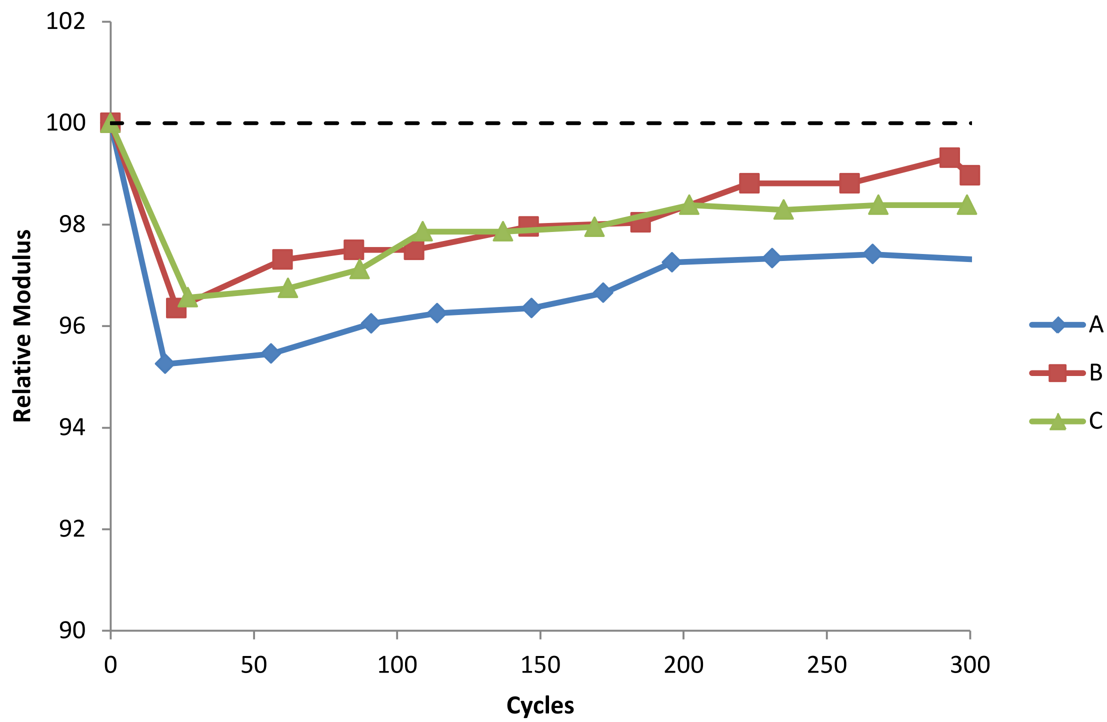

FIGURE 5.9
7 Day Lime Water Cured Relative Modulus with Different Aggregates

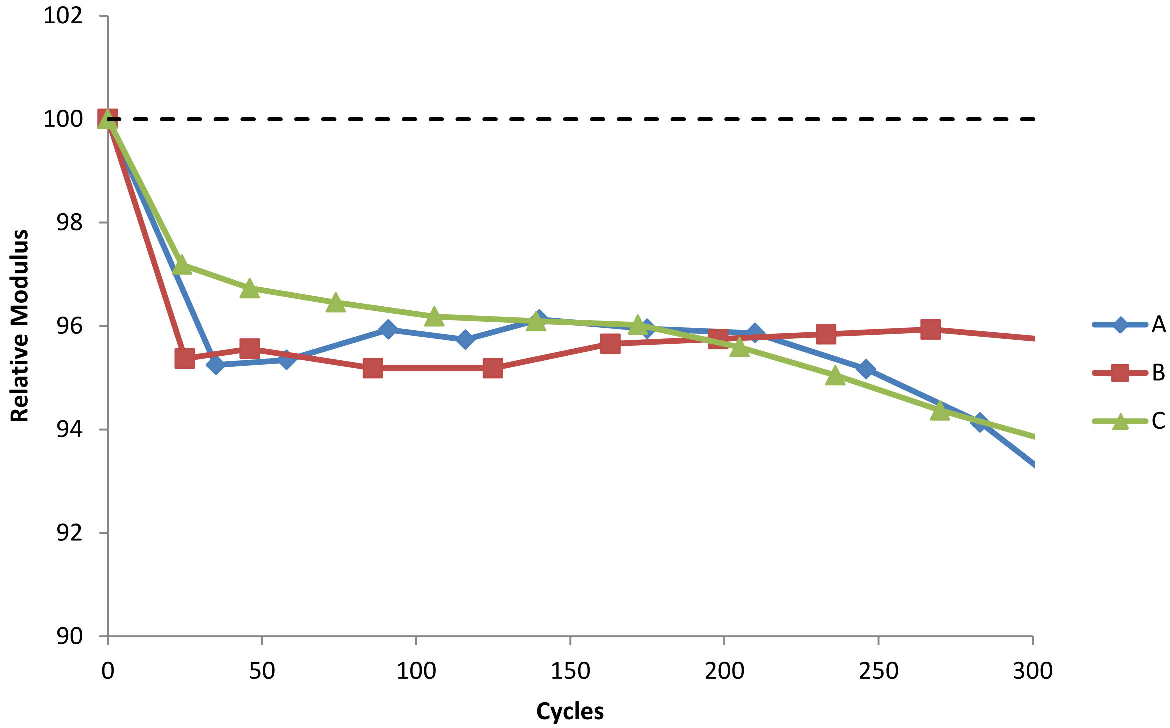

FIGURE 5.10
14 Day Lime Water Cured Relative Modulus with Different Aggregates

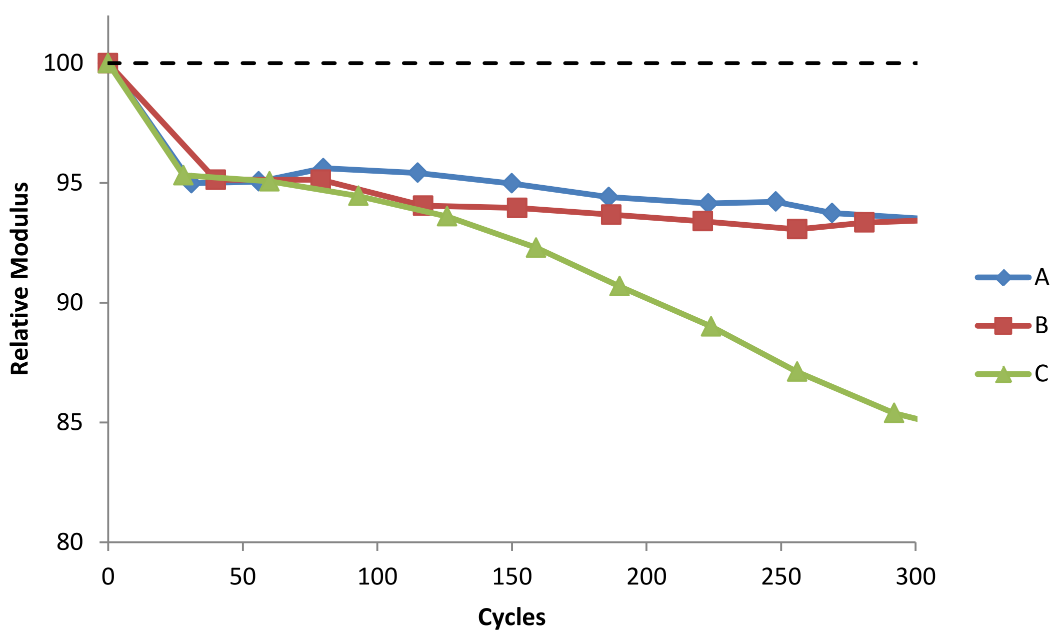

FIGURE 5.11
21 Day Lime Water Cured Relative Modulus with Different Aggregates

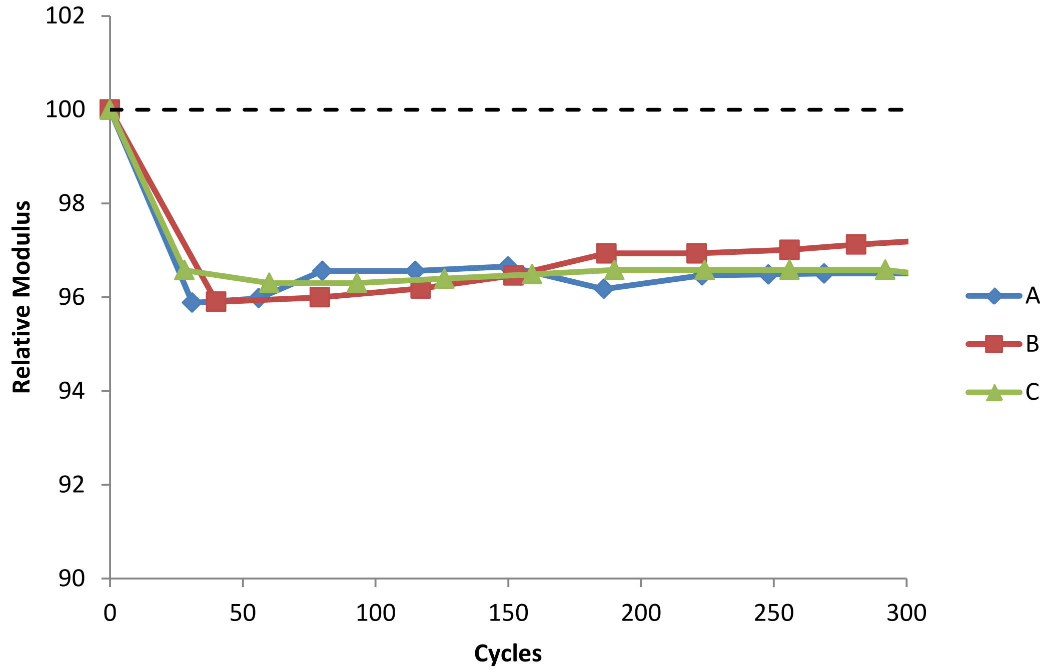

FIGURE 5.12
28 Day Moist Cured Relative Modulus with Different Aggregates

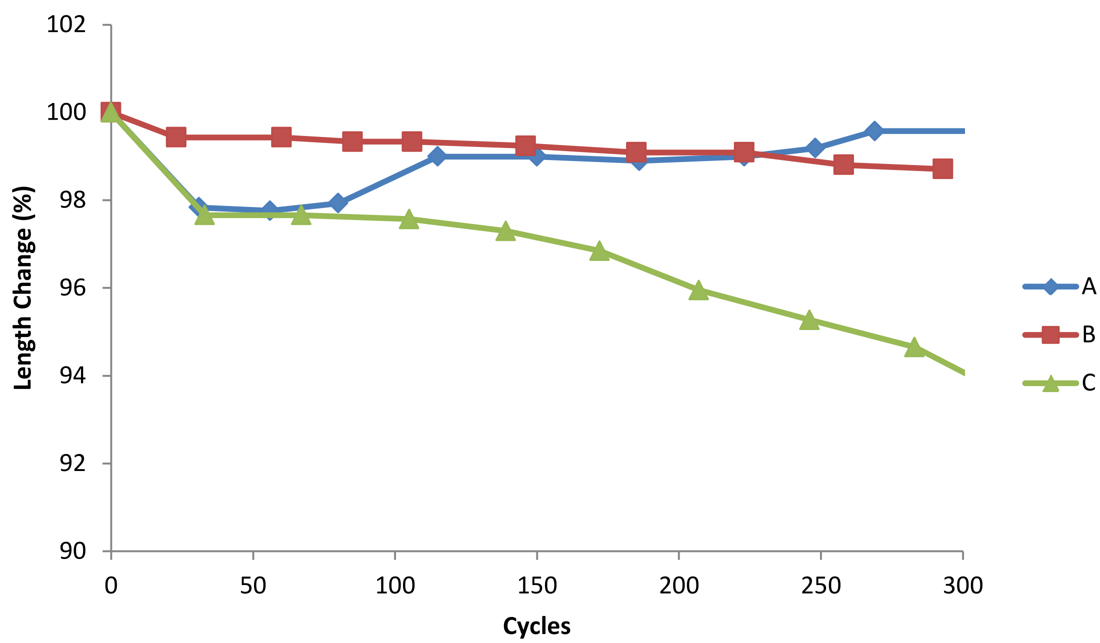

FIGURE 5.13
Standard Cured Relative Modulus with Different Aggregates

The good performance of concrete samples made with concrete from aggregates A and B and cured according to standard KTMR-22 or 28 days in the 100% moist room showed that the paste is strong enough to withstand freeze-thaw deterioration after shorter curing periods. This contradicts the assumption that the 67 days of curing in the 100% moist room is needed to ensure that the paste strength does not cause an otherwise good aggregate to fail KTMR-22.

The curing method used was shown to affect the durability in freeze thaw testing conditions. The KTMR-22 curing method was shown to be less severe than all of the curing methods that did not involve a drying period. For the 7 day and 14 day lime water cured and 100% moist room cured concrete samples, the results were similar for all aggregates tested. The standard curing method and 21 day curing in lime water showed more deterioration in aggregate C, although curing in a lime water bath for 21 days is more severe.

Higher rates of deterioration would be expected for concrete that has a higher level of saturation. The concrete water absorption level was measured in order to better understand how the different curing methods are affecting the severity of damage experienced during freezing and thawing cycles. Three concrete samples that were cured using curing method 3 (cured up to 7 days of age in the 100% moist room followed by 21 days of curing at 100°F in a lime water bath) were weighed periodically during the curing period. It was not necessary to measure the weight of separate samples for curing methods 1 and 2, since the methods are the same as method 3 until the curing is terminated and freeze-thaw cycles are started. Two concrete samples cured using the standard KTMR-22 curing methods were weighed periodically throughout the curing period. Since the curing for KTMR-22 and method 4 are the same until 28 days of age when the samples cured according to method 4 were removed from the 100% moist room, separate weight measurements were not taken for samples cured according to method 4.

The average mass change for the concrete specimens for the standard KTMR-22 curing method and method 3 curing is shown in Figure 5.14. It was seen that the weight increase for the concrete samples during curing was similar for lime water soaking or curing in the 100% moist room. The drying cycle however caused the concrete to lose mass to below that seen after removal from the forms. After the concrete is placed in water before the freeze thaw cycles, it reabsorbs water at a higher rate than seen immediately after form removal. The weight increase at the end of curing for method 1 (7 days in the 100% moist room, 7 days in the lime water bath) would be the same as the weight increase for method 3 after 7 days in the 100% moist room and 7 days in the lime water bath. The weight increase at the end of curing for method 2 (7 days in the 100% moist room, 14 days in the lime water bath) would be the same as the weight increase for method 3 after 7 days in the 100% moist room and 14 days in the lime water bath. The weight increase for concrete cured according to method 4 (28 days in the 100% moist room) would be the same for concrete cured according to KTMR-22 after 28 days of curing.

Table 5.1 shows the weight gain at the end of the curing period for each of the 5 curing methods. For the concrete cured in the lime water bath at 100°F, the longer the curing period and higher degree of saturation resulted in higher amounts of damage. The samples cured at 100°F in the lime water bath for 21 days showed a higher rate of deterioration than the samples cured at 28 days in the 100% moist room, even though their water absorbed should be similar. This may be because the higher temperature changed the paste pore structure, reducing the concrete resistance to freeze-thaw. The lower damage seen in the standard KTMR-22 testing can be explained by the lower concrete degree of saturation seen in KTMR-22 compared to the other 4 curing methods used. This supports the theory that the drying period used during curing according to KTMR-22 lengthens the number of freeze-thaw cycles needed to differentiate poor from good performing aggregates by reducing the concrete degree of saturation at the beginning of the test.

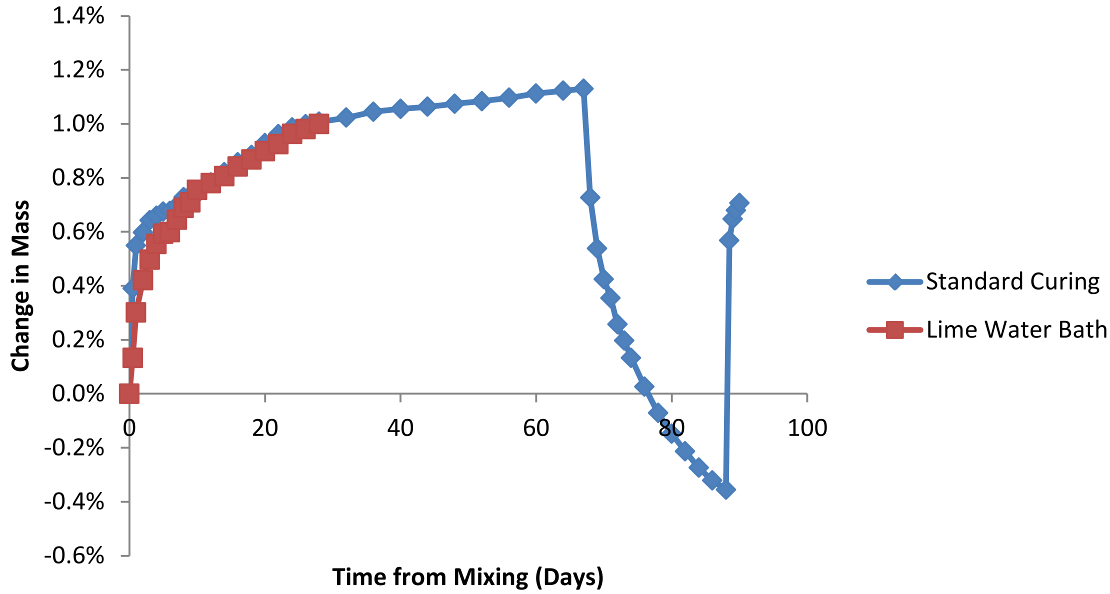

FIGURE 5.14
Change in Mass for Concrete Samples Exposed to Different Curing Methods

TABLE 5.1
Theoretical Mixture Proportions

Curing Method	Curing Method Description	Weight Gain at End of Curing Period (%)
1	7 days 100% moist room, 7 days in 100°F lime water bath	0.81
2	7 days 100% moist room, 14 days in 100°F lime water bath	0.91
3	7 days 100% moist room, 21 days in 100°F lime water bath	1.0
4	28 days in 100% moist room	1.0
KTMR-22	67 days in 100% moist room, 21 days in 50% relative humidity chamber at 73°F	0.71

5.3 Task 3

Task 3 investigated the ability to increase the freeze-thaw durability of concrete containing nondurable aggregates by varying the paste permeability. The experiments performed changed the w/cm and SCMs used to vary the paste permeability. Three different aggregates were used to make concrete specimens for this task: aggregate A, aggregate C, and aggregate D. These mixtures used concrete with different w/cm and SCMs to see if reducing the water ingress could reduce the deterioration rate of concrete. Testing for this task included freeze-thaw durability, rate of absorption, chloride resistivity, and measuring the permeable pore space of the concrete. Concrete curing used in this task was as specified by KTMR-22.

5.3.1 Concrete Rapid Freezing and Thawing Testing

The relative modulus results for the freeze-thaw testing conducted during Task 3 are shown in Figure 5.15 through 5.17. Figures 5.18 through 5.20 show the percent length change during the freeze-thaw cycles. The specimens made with aggregate A were only tested to 300 cycles as these tests were completed before the research team learned that the number of freeze-thaw cycles specified by KTMR-22 might increase.

The relative modulus of elasticity shows that the beams for aggregates A and D performed well with none of the mix designs testing below 90% of the initial modulus. The mixtures containing aggregate A and the ternary blend mixture or fly ash C2 fell below the KDOT durability factor threshold of 95. The cause of this is unknown, however it is possible that some specimen or mixture variability was the cause. Aggregate C showed significantly more deterioration than aggregates A and D. For aggregate C, three mixture designs that had a durability factor at or below 70. These were the 0.39 w/cm mix, the 0.45 w/cm mix, and the mix with C2 as the SCM. The C2 mix failed before the 600 cycles were up. It failed with a durability factor of 56 after only 600 cycles. Both the 0.39 w/cm and 0.45 w/cm mixes survived the 600 cycles. At 600 cycles, the 0.39 w/cm mix had a durability factor of 43 and the 0.45 w/cm mix had a durability factor of 70. All other mixes were in the low to mid 80s range with the exception of the ternary blend mix, which had a durability factor of 91 after 600 cycles. Like the modulus of elasticity testing results, both the aggregate A and D specimens performed well. Aggregate C showed very large expansions near 1%, matching the damage seen in the relative modulus measurements.

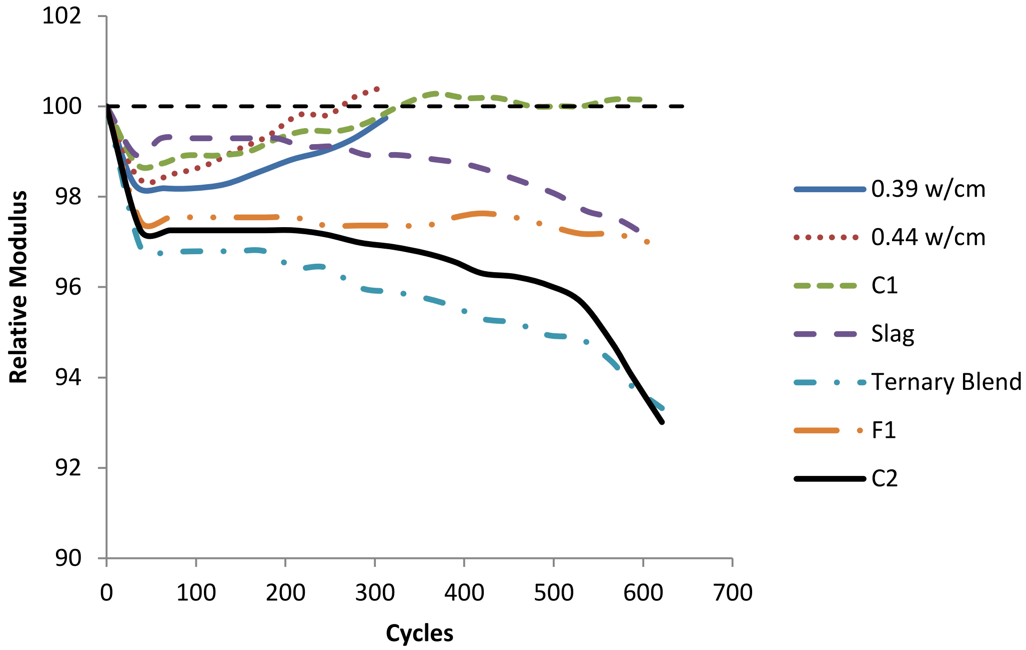

FIGURE 5.15
Aggregate A Relative Modulus

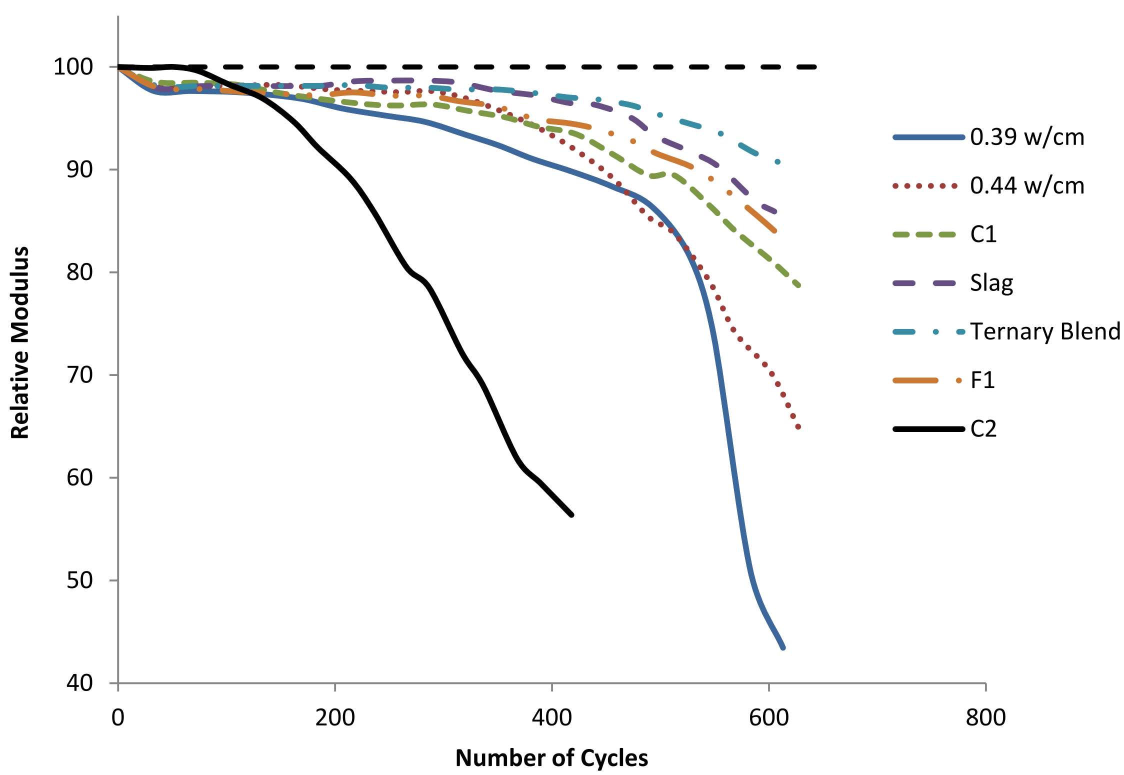

FIGURE 5.16
Aggregate C Relative Modulus

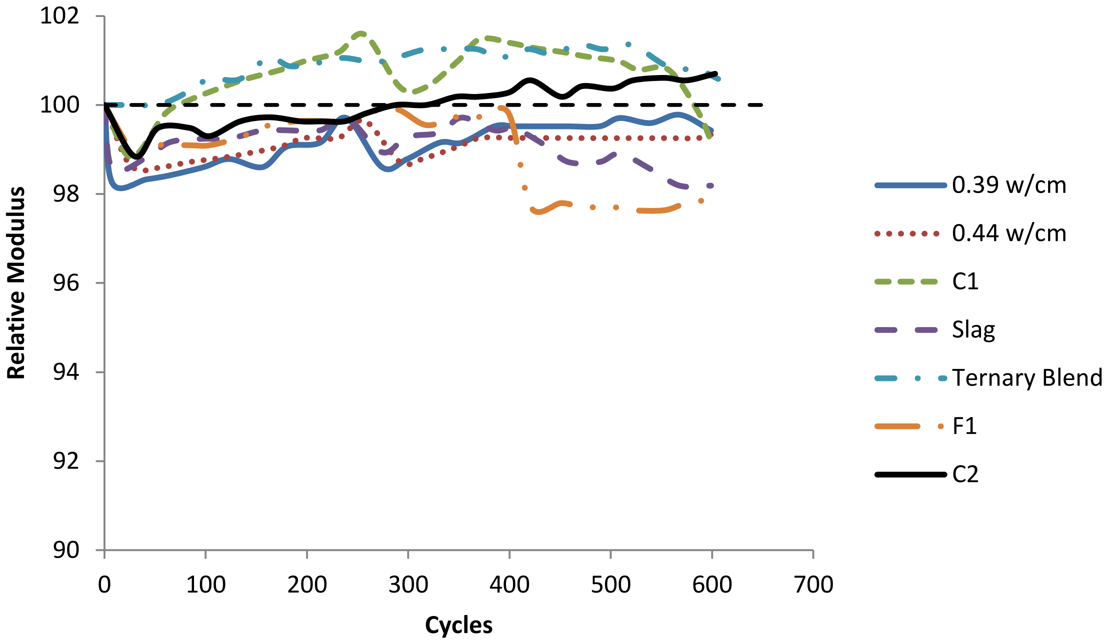

FIGURE 5.17
Aggregate D Relative Modulus

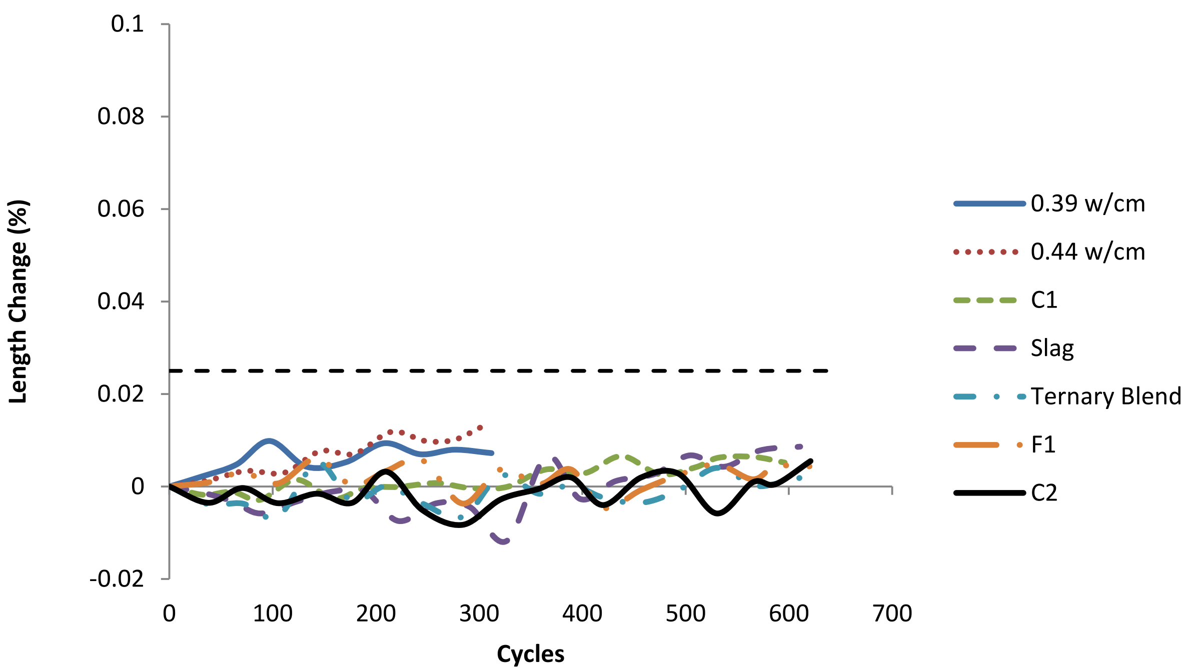

FIGURE 5.18
Aggregate A Length Change Measurements

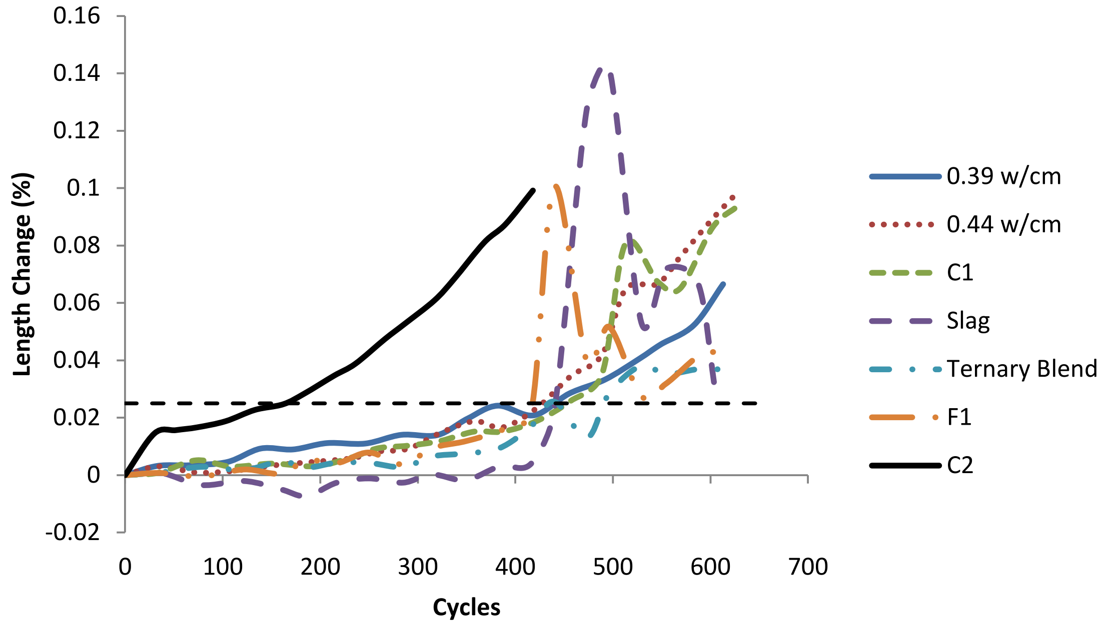

FIGURE 5.19
Aggregate C Length Change Measurements

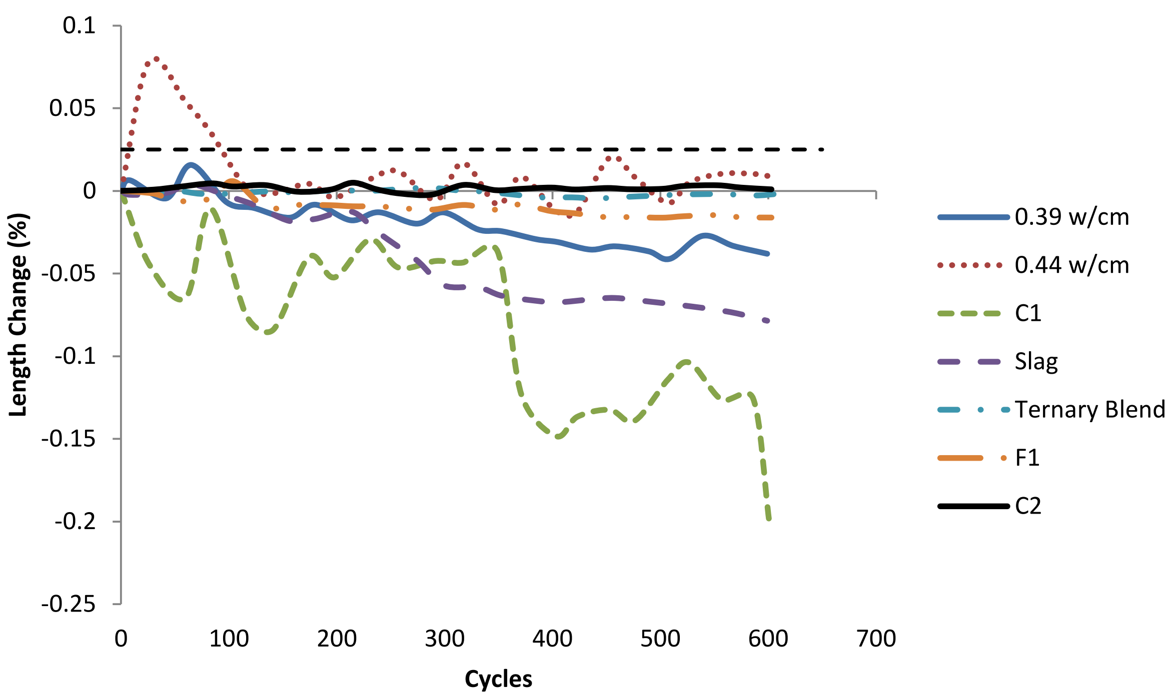

FIGURE 5.20
Aggregate D Length Change Measurements

5.3.2 Concrete Resistance to Water Ingress

The rate of absorption shows the rate at which water was able to infiltrate into the concrete. The initial and secondary absorption rates for concrete made with aggregates A, C, and D are shown in Figures 5.21 through 5.22, respectively. The concrete rapid chloride permeability for concrete made with aggregates A, C, and D is shown in Figure 5.23. The volume of permeable voids for concrete made with aggregates A, C, and D is shown in Figure 5.24. Some differences were seen between the calculated initial absorption values for concrete made with the same cementitious materials and different aggregates, however the secondary absorption values which were less variable. This difference may be because of small differences in the first absorption measurements from normal variability may change the calculated parameters. The most consistent water ingress parameter between mixtures made with the same cementitious materials but different aggregates was the volume of permeable voids.

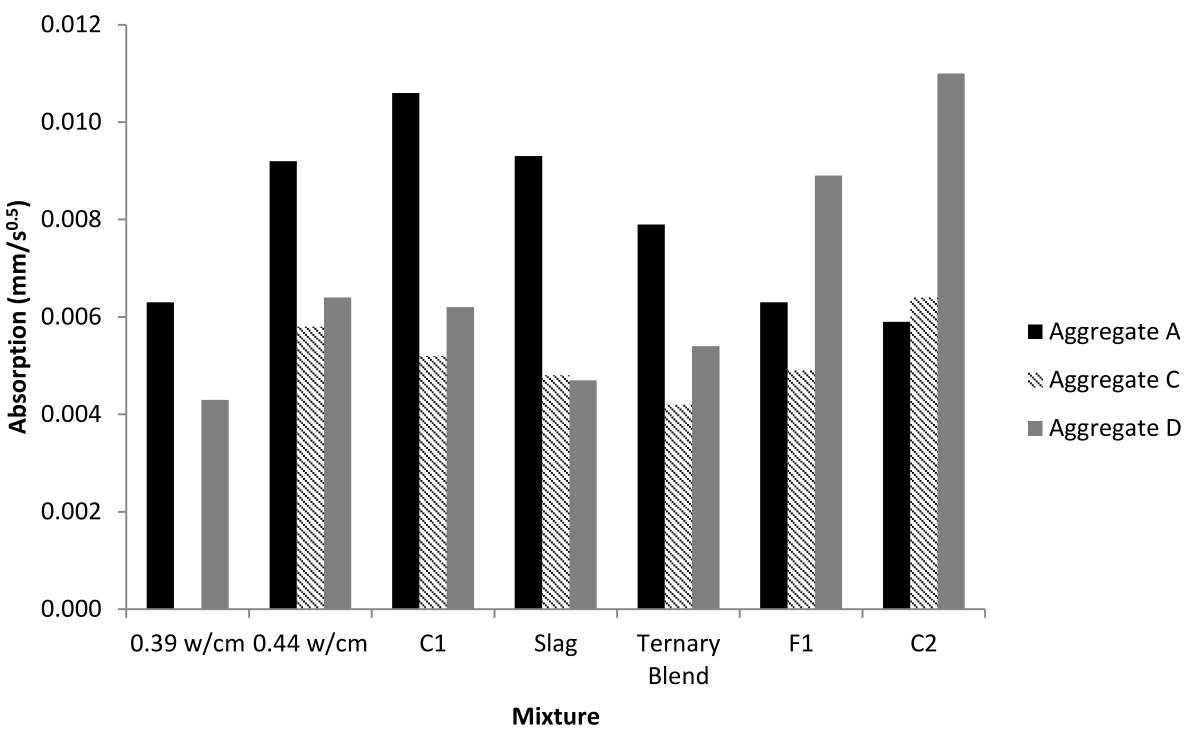

FIGURE 5.21
Concrete Mixture Initial Absorption

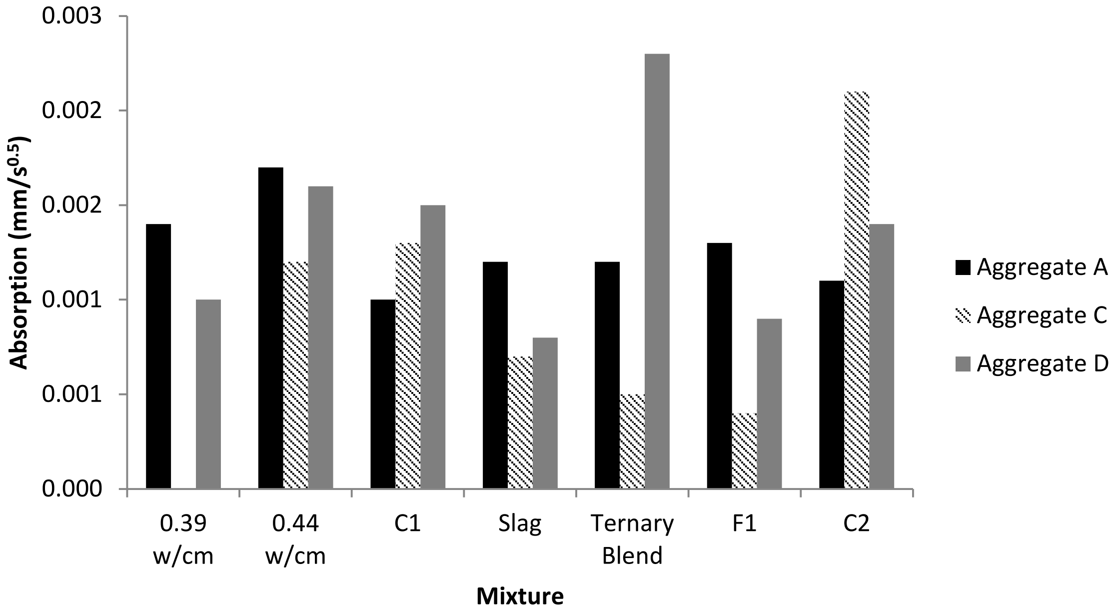

FIGURE 5.22
Concrete Mixture Secondary Absorption

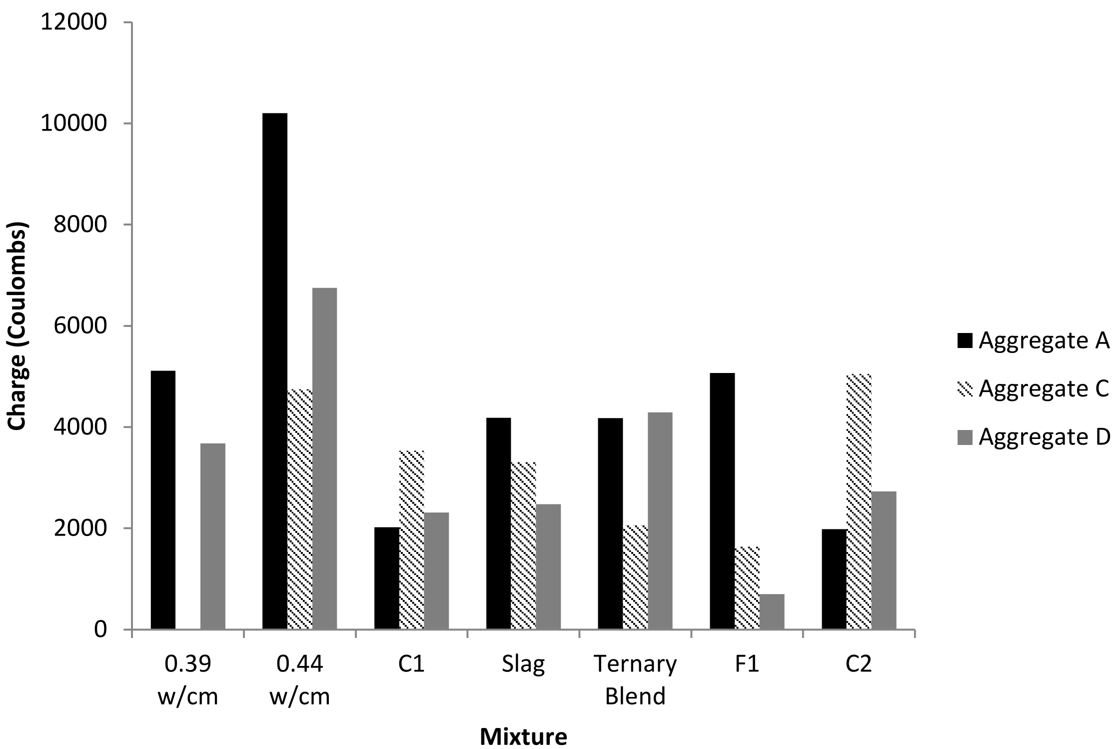

FIGURE 5.23
Concrete Mixture Rapid Chloride Permeability (ASTM C 1202) Results

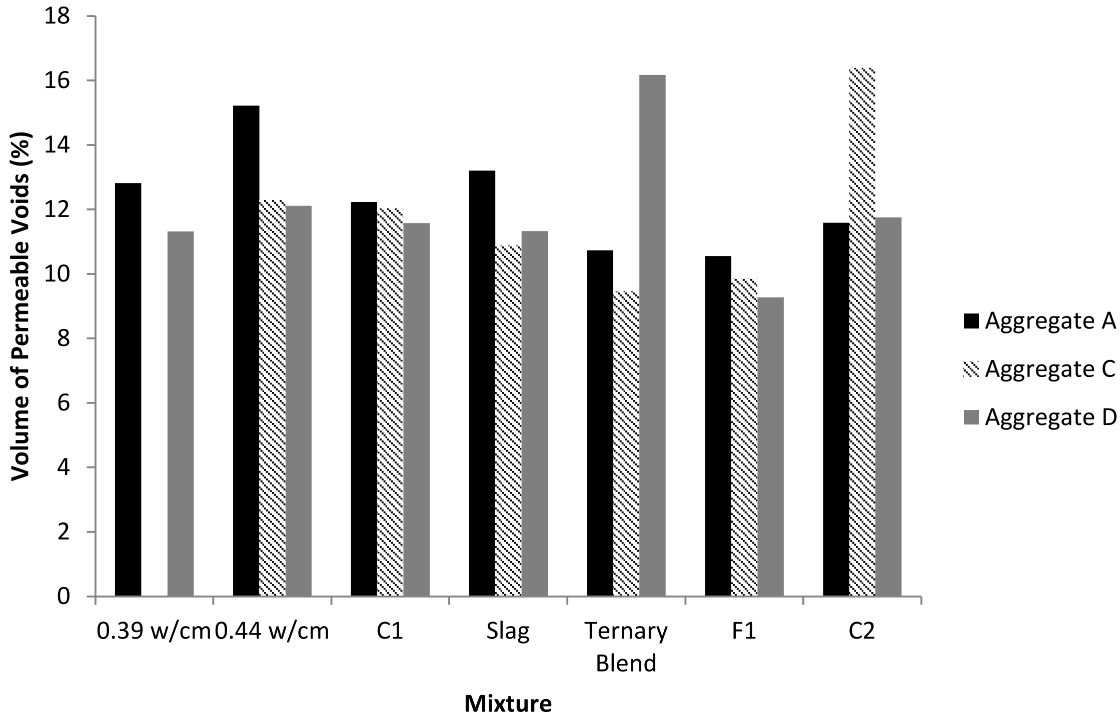

FIGURE 5.24
Concrete Mixture Volume of Permeable Voids (%)

A comparison was made between the concrete durability in freeze thaw seen with the measures of water penetration resistance used for Aggregate C. Figure 5.25 shows a comparison of the durability factor at 600 cycles versus the volume of permeable voids. Figure 5.26 shows a comparison of the durability factor at 600 cycles versus the rapid chloride permeability. Figure 5.27 shows the durability factor at 600 cycles versus the absorption coefficients.

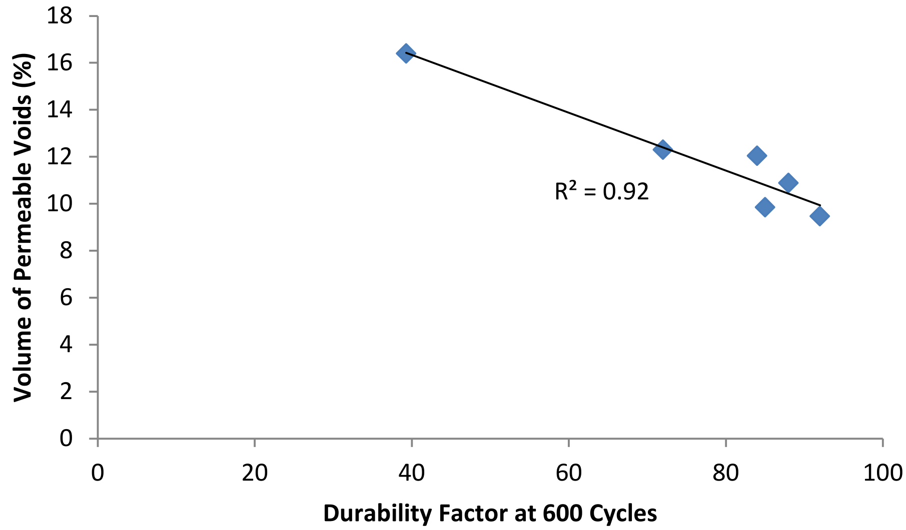

FIGURE 5.25
Comparison of Concrete Durability Factor versus Volume of Permeable Voids for Concrete Made with Aggregate C

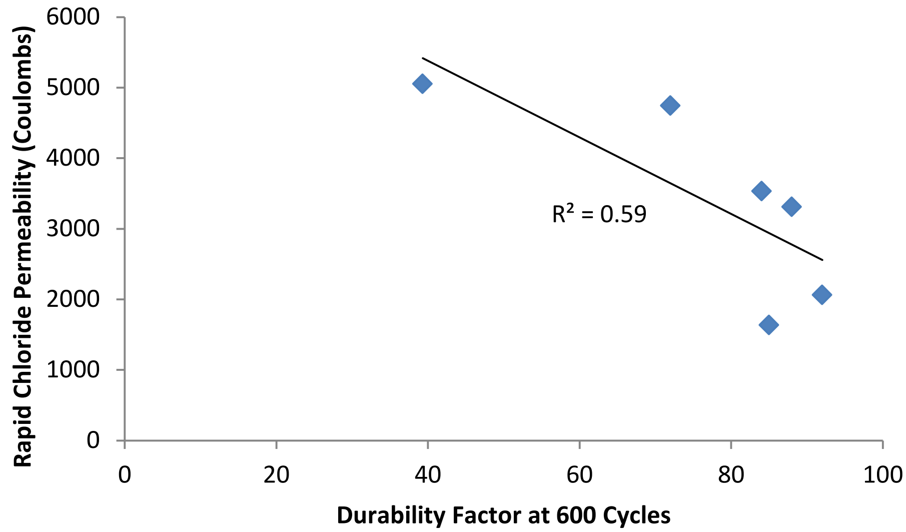

FIGURE 5.26
Comparison of Concrete Durability Factor versus Rapid Chloride Permeability for Concrete Made with Aggregate C

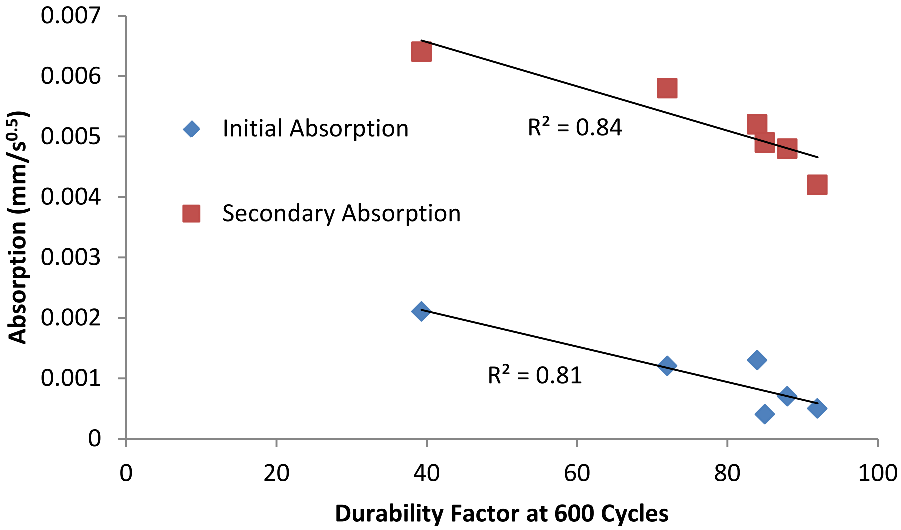

FIGURE 5.27
Comparison of Concrete Durability Factor versus Absorption Coefficients for Concrete Made with Aggregate C

While the use of SCMs was able to improve the concrete resistance to freeze-thaw damage, none of the concrete mixtures tested with concrete made with aggregate C were able to maintain a durability factor of 95 past 600 cycles. This means that the addition of SCMs in concrete would not justify allowing the use of nondurable aggregates in concrete pavements. The use of SCMs in concrete should however be encouraged because they could extend by a few years the life of pavements when substandard aggregates are unknowingly or inadvertently used, besides any other benefits of SCM use in concrete.

A high degree of correlation was seen between the concrete volume of permeable voids and durability factor at 600 cycles for aggregate C. Additionally, the concrete absorption coefficients correlate well with the concrete freeze thaw durability. This is most likely because the decreased absorption rate seen by some mixtures reduced the rate of water ingress, decreasing the saturation level of the concrete and decreasing the concrete deterioration during freezing. The durability factor did not correlate as well however with the rapid chloride permeability. This is most likely because of the high variability expected with the rapid chloride permeability test method. The increased number of cycles until damage occurred in the concrete specimens made with SCMs implies that concrete pavements made with SCMs would take more freezing and thawing exposure cycles in the field for damage to appear than concrete made without SCMs. The freeze thaw results however also imply that the use of SCMs will not delay freeze-thaw damage in concrete pavements made with poor quality aggregates enough to meet KDOT service life requirements. The enforcement of current KDOT specifications used to limit the concrete volume of permeable voids is likely to help extend the life of concrete pavements, especially when SCMs that reduce permeability are used. The use of high quality SCMs may also be beneficial to the concrete in preventing alkali-silica reaction, reduce the heat of hydration, and increase long term strength and is recommended.

If a durability of 95 after 660 cycles is assumed to correspond to a 20 year service life in the field, and a linear damage model was assumed, every 33 cycles of freezing and thawing would correspond to 1 year of pavement service life. Using this service life assumption, the use of a 0.39 w/cm with concrete made using aggregate C would correspond to a service life of 7.5 years of pavement service life. The use of a ternary blend at 0.39 w/cm with aggregate C would then correspond to a service life of 15 years. This increase in potential service life could help justify the use of SCMs in concrete pavements and continued enforcement of concrete permeability specifications as a secondary level of safeguarding against premature failure in the case that nondurable aggregates are used inadvertently because of unanticipated changes in a quarry bed, but not as a primary d-cracking mitigation measure. Quarry monitoring and aggregate quality control testing to ensure that only freeze-thaw durable aggregates are approved for and used in Kansas concrete pavements are needed to ensure that Kansas concrete pavements meet the expected 20 year service life.

Chapter 6: Conclusions and Recommendations

6.1 Conclusions

In this study, the role of concrete curing, mixture proportioning and presence of SCMs in the mixture, and aggregate type on the freeze thaw durability of concrete beams tested using ASTM C 666 method B were investigated. The methods used by the researchers were validated through comparisons of equipment measurements on the same samples and companion beams tested at KSU and KDOT. The experimental results obtained have led to the following conclusions:

1. The equipment and methods used by KSU follow KDOT specifications. KSU researchers found that the impact resonance measurements could match those made by KDOT, giving confidence in the equipment and methods used by KDOT. ASTM C 666 does not contain multi-laboratory precision information, however results obtained by KSU and KDOT were just outside of the range expected for acceptable within-laboratory tests.
2. The good performance of concrete samples made with concrete from aggregates A and B and cured according to standard KTMR-22 or 28 days in the 100% moist room showed that the paste is strong enough to withstand freeze-thaw deterioration after shorter curing periods. This contradicts the assumption that the 67 days of curing in the 100% moist room is needed to ensure that the paste strength does not cause an otherwise good aggregate to fail KTMR-22.
3. Curing methods used on concrete beams can greatly affect the freeze thaw durability. The drying period used by KTMR-22 was shown to decrease the severity of the test during the freezing and thawing cycles. The drying period used by KDOT during curing increases the water reabsorption rate during concrete soaking and tempering prior to beginning freeze thaw cycles. This concrete degree of saturation after the drying period and subsequent retempering in water baths for 48 hours prior to beginning freeze-thaw cycles was still lower however, than the shorter curing periods that did not involve drying periods. The increased degree of saturation seen by longer periods of curing in lime water showed

increased levels of damage in concrete containing non-durable aggregates than seen by standard KTMR-22 curing. The results show that if the KTMR-22 curing methods were replaced with a shorter curing period using either the accelerated curing methods used in this study in lime water or 28 day curing in a 100% moist room, the test would be made slightly more severe and would decrease the total time needed to perform KTMR-22 by at least 2 months.

4. It was seen that the concrete volume of permeable voids and water absorption rate correlated well with the freeze thaw durability of concrete made with a poor quality aggregates. The use of SCMs was able to reduce the rate of freeze-thaw deterioration through a reduction in the water ingress rate. This reduction in the water ingress rate was not enough however, to transform a concrete containing non-durable aggregates into a freeze-thaw durable concrete mixture as measured by KTMR-22.

6.2 Implementation Recommendations

The following recommendations for implementation are made based on the experiments conducted in this study:

1. Change the curing regime used in KTMR-22 to 28 days of curing in a 100% moist room. Curing in a 100% moist room for 28 days instead of the standard KTMR-22 curing method consisting of 67 days in the 100% moist room followed by 21 days of drying at 50% relative humidity would reduce the total time needed to determine the acceptability of an aggregate for use in concrete by 60 days and make the test slightly more severe. Curing in a 100% moist room is also believed to be simpler to implement at KDOT facilities than curing at 100°F in lime water.
2. Continue enforcement of KDOT permeability specifications. Although the use of SCMs and lower concrete permeability cannot make concrete containing nondurable aggregates acceptable for use in pavement applications, it could serve as a secondary safeguard to help limit the decrease in service life associated with inadvertent use of nondurable aggregates.

3. Focus on quarry monitoring and aggregate quality control testing to ensure that only freeze-thaw durable aggregates are approved for and used in Kansas concrete pavements.

6.3 Future Research

Several potential areas of future research on aggregate freeze-thaw durability of concrete pavements exist. Some of these areas are as follows:

1. Determine if the number of freeze-thaw cycles needed to determine adequate performance in KTMR-22 can be reduced by the use of ASTM C 666 method A instead of method B as currently used. In method A, the concrete is immersed in water during the freezing and thawing cycles. In method B, the concrete is frozen in air, allowing the concrete beams to dry. The higher concrete degree of saturation from method A may reduce the number of cycles needed for damage to appear.
2. Measure the air flow speed, relative humidity, and temperature rates for different size concrete freeze-thaw chambers to determine how if at all the chamber size and number of specimens in the chamber influences the freezing rate and drying that occurs in ASTM C 666 method B during the freezing period.

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